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ALTERNATIVE APPROACHES FOR UPGRADING EFFLUENT QUALITY FOR LAGOON BASED SYSTEMS

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EXECUTIVE SUMMARY

In order to establish Best Available Technology Economically Achievable (BATEA) in rural and semi-rural communities which presently use conventional lagoon technology, a need exists to evaluate alternatives which can be used to improve lagoon effluent and establish costs for introducing such alternatives. Two such methods for improving lagoon effluent quality which are presently being used by some Ontario lagoon facilities are the "Sutton" and "New Hamburg" concepts. The Sutton concept consists of a nitrifying extended aeration plant followed by polishing lagoons, with waste sludge discharged into the lagoons. The New Hamburg concept consists of any number of aerated or facultative lagoons, with the lagoon effluent sprayed intermittently over sand filters.

Conventional facultative lagoons are subject to significant seasonal effects due to climatic conditions. This is demonstrated by elevated ammonia and H_2S concentrations and toxic effects in the effluent during the spring and winter discharges as well as elevated TSS concentrations during the spring and fall discharges for annual and seasonal lagoons. Many lagoons also have difficulty meeting monthly phosphorus limits of 1.0 mg/L.

The Sutton Process plants produce an improved effluent quality relative to conventional facultative lagoons in terms of BOD_5 and TSS concentrations, based upon operational results of several existing Sutton process facilities in Ontario. Seasonal increases in TSS concentrations, as observed in conventional lagoons, are not evident in Sutton Process plants, and H_2S production in winter months is also avoided by operating in this mode. Insufficient data are available to allow conclusions to be drawn in terms of toxicity reduction relative to conventional lagoons. Significant reduction in ammonia concentrations are achieved in the initial period of Sutton process operation. However, increases in ammonia concentration across the polishing pond has been observed in the first years of Sutton Process operation, suggesting that ammonia is being solubilized from the sludge pile in the pond over time. The effects on effluent quality are more dramatic after 5-7 years of plant operation, suggesting a need to implement a regular program of sludge removal from the lagoon. A wide variability in design approach and associated cost was observed, with a

move towards more conservative designs with the more recent examples of Sutton Process facilities.

Although there are limited data upon which to base an assessment of the New Hamburg Process in Ontario, available information suggests that the use of intermittent sand filters results in a significant improvement in effluent quality in terms of BOD_5, TSS, TP, TKN, NH_3-N and H_2S concentrations. The limited toxicity data suggests that a reduction in acute lethality is accomplished across the filter in the spring. Seasonal night-time operation of the filter is necessary in order to avoid the problems of excessive vegetative growth and winter freeze up across the filter surface.

Based on available data from facilities currently operating in Ontario and the statistical procedures adopted by the MISA (Municipal Industrial Strategy for Abatement) program, the monthly effluent limits for the Sutton and the New Hamburg processes were found to be as follows:

EFFLUENT QUALITY (mg/L)

PROCESS	BOD_5	TSS	TP	NH_3-N
Sutton	15	15	1.0	4.0
New Hamburg	5	10	1.0	4.0

Costs for new and retrofit Sutton and New Hamburg facilities for various sized plants are estimated as follows:

LIFECYCLE COSTS ($/YR)*

PLANT TYPE	DESIGN SIZE		
	300 m^3/d	1000 m^3/d	3300 m^3/d
Sutton Plants			
• New	200,000	320,000	560,000
• Retrofit	170,000	250,000	420,000
New Hamburg Plants			
• New	140,000	240,000	530,000
• Retrofit	90,000	120,000	200,000

* _ 1992 Dollars, ENR Index = 6537. The costs include amortized capital (20 years at 8% interest rate) and operating and maintenance costs.

The approximate capital costs for upgrading the existing conventional lagoons (seasonal, annual and continuous discharge) in Ontario to Sutton and New Hamburg process facilities are estimated at $265 million and $112 million, respectively.

1.0 INTRODUCTION AND PROJECT OBJECTIVES

1.1 Background

The MISA program (Municipal Industrial Strategy for Abatement) began in 1987, with the goal of virtual elimination of toxic chemicals discharged to surface waters. This included the establishment of Best Available Technology Economically Achievable (BATEA) effluent limits for both industrial and municipal sector sewage treatment plants. To establish BATEA, three major steps must be followed:

- review effluent quality achievable by various Best Available Technologies (BATs);

- define the costs (capital and operations) of applying the BATs to new treatment facilities and to retrofit existing facilities;

- analyze the economic impacts of imposing effluent limits achievable by the various BATs.

Lagoon based sewage treatment systems have historically been used in rural and semi-rural communities as a low cost alternative to conventional secondary sewage treatment. In Ontario, there are about 150 lagoon treatment systems presently operating. This group includes continuous, annual and seasonal discharge systems, both with and without aerated cells, many with the provision for phosphorus removal through chemical addition. Regardless of the lagoon system, lagoons can achieve at best secondary quality effluent. In order to establish BATEA in rural and semi-rural communities, a need exists to evaluate alternatives which can be used to improve lagoon effluent and establish costs for imposing such alternatives.

Two such alternatives for improving effluent quality in lagoon based systems are the "Sutton" and "New Hamburg" concepts, each named after the facility in the respective

town where the concepts were first introduced. The Sutton concept consists of an aerated cell(s) and clarifiers, followed by polishing lagoons, and operates in a continuous discharge mode. Waste sludge is discharged into the polishing lagoon. The New Hamburg concept consists of any number of aerated or facultative lagoons, with the lagoon effluent being sprayed over sand filter beds, and operates in seasonal discharge mode.

Presently, two sewage treatment plants in the Province use the New Hamburg concept, while 8 use the Sutton concept. Much concern has been expressed towards the application of these and future planned installations, because although they appear to produce a tertiary quality effluent, an investigation has yet to be made to determine if they are truly BATEA plants.

R.V. Anderson Associates Limited, in conjunction with XCG Consultants, have been retained by the Ministry of the Environment and Environment Canada to carry out this investigation. Other organizations which were retained as sub-consultants on this project included Beak Consultants, for toxicity evaluations, and Environmental Engineering Consultants (of Norwich, Vermont), for a review of alternative lagoon technologies in the U.S.

This project was carried out under the guidance of a steering committee comprised of representatives of the Ministry of the Environment, Environment Canada, and the Municipal Engineers's Association.

1.2 Objectives

The objectives of this project are to evaluate existing performance data of various lagoon based sewage treatment systems throughout Ontario, especially those using the Sutton and New Hamburg processes. The review of the processes included effluent quality, design, operation, maintenance and cost considerations. The results of the evaluation give an indication as to whether or not the Sutton and New Hamburg processes can be

efficiently and economically applied as a technology to produce tertiary quality effluent, especially in situations where existing lagoon based systems require this quality but are unable to achieve it.

1.3 Study Scope

The assessment of the performance capabilities of conventional lagoons, Sutton Process plants and New Hamburg Process plants was based on available information collected from the MOE and the plant operating authorities. No process monitoring or sampling was undertaken in conjunction with this assessment with the exception of sampling and analysis of sludges accumulated in the Sutton lagoons at the Sutton WPCP, Colborne WPCP and Tottenham WPCP. This sampling and analysis were undertaken by MOE staff during the fall and winter of 1991/92.

The historic review of conventional lagoon design and performance was based on information contained in the MOE Utility Monitoring Information System (UMIS) database. As no readily available source of performance or design data was available for Federally owned/operated lagoon systems, these facilities were excluded from the review. The UMIS database for provincially regulated systems was reviewed for the period from 1986 through 1990. The database included information on the type of lagoon (continuous discharge, annual discharge, seasonal discharge, aerated lagoon, etc.), location (MOE Region), phosphorus removal capability (continuous, batch or no chemical addition), the design capacity, the average daily flow, raw sewage and final effluent quality (BOD_5, TSS, TP, TKN, NH_3-N, NO_3-N, NO_2-N), and effluent loadings to the receiving stream. The study team did not verify the design or performance data contained in the UMIS database. No up-to-date data on H_2S concentrations in conventional lagoon effluents were available for review.

The evaluation of the Sutton Process plants and New Hamburg Process plants was based on performance data contained in the UMIS database for these facilities from the entire period of their operation. Since the UMIS database did not contain analytical results for

intermediary streams within the plant (extended aeration plant effluent in Sutton Process plants or filter influent in New Hamburg Process plants), these data were acquired from the operating authority. Each of the operational Sutton and New Hamburg Process plants was visited as part of the project and the operation and performance discussed at length with the operating staff. At that time, plant operating logs were acquired and additional design, operating and performance information collected for subsequent analysis. In addition, the MOE had undertaken extensive sampling of the Sutton WPCP (1981 through 1986), the Colborne WPCP (April 1986 through June 1987) and the New Hamburg WPCP (March/April 1991 and March 1992). The results of these monitoring programs were included in the data analysis for these facilities where appropriate.

Limited effluent toxicity data were available on which to base an analysis of the performance of conventional lagoons, Sutton Process plants and New Hamburg Process plants. Effluents from two continuous discharge lagoons had previously been assessed for acute and chronic toxicity effects (Beak, 1991). In addition, MOE provided acute toxicity data for effluents from four of the Sutton Process plants and from the New Hamburg WPCP. These data were generated in the summer of 1990 and the winter of 1991.

1.4 Report Format

This report presents the findings of the evaluation of the Sutton Process and New Hamburg Process as low cost alternatives for upgrading lagoons in small municipalities.

Following this introductory section, conventional lagoon treatment systems are described in Section 2.0. The design philosophy and operating strategy currently applied in Ontario for continuous discharge lagoons, seasonal discharge lagoons and annual discharge lagoons (fill-and-draw facultative lagoons) are discussed. The performance capabilities of these systems based on historic data for Ontario facilities are presented. Estimated capital and operating cost data for these types of facilities are also provided.

In Section 3.0, the Sutton Process and New Hamburg Process technologies are described. A summary of the design, operation and performance history of existing examples of these processes in Ontario are presented, along with operating and performance problems identified as a result of this review. Capital and operating cost information for the existing examples are also provided. More detailed design, operation and performance data for each existing Sutton Process plant and New Hamburg Process plant in Ontario are presented in Appendix 1 (Sutton Process plants) and Appendix 2 (New Hamburg Process plants). This section concludes with a discussion of the advantages and disadvantages of Sutton and New Hamburg Process technologies relative to conventional lagoon treatment technologies. A brief discussion of other approaches available to upgrade lagoon treatment systems to produce improved effluent quality in terms of effluent ammonia, H_2S and toxicity is also presented based on a technical review undertaken by Environmental Engineering Consultants of approaches which have been tried in the United States in particular. The more detailed report describing other lagoon upgrading alternatives is provided as Appendix 3. Appendix 4 contains the supporting costing information.

Section 4.0 presents design and cost data for new and retrofit Sutton and New Hamburg Process plants based on the data collected and reviewed in this investigation and the design criteria recommended in this report. The costs of Sutton and New Hamburg Process technologies are compared with the costs of conventional lagoon treatment technologies.

This investigation identified some specific areas where additional information is required before the suitability of these process technologies for widespread use to upgrade conventional lagoons in Ontario can be recommended. These information gaps are identified and discussed in Section 5.0 of the report.

Conclusions and recommendations resulting from this investigation are presented in Sections 6.0 and 7.0, respectively.

2.0 CONVENTIONAL LAGOON TREATMENT TECHNOLOGY

2.1 General

Based on the 1990 UMIS database, there are 137 municipal sewage lagoons in Ontario which discharge either continuously or intermittently to the receiving water. Table 2.1 provides a summary of the distribution by treatment technology and location for these 137 facilities. This total excludes those facilities which dispose of their treated effluents by spray irrigation or by exfiltration as footnoted in Table 2.1 as well as lagoons which have been upgraded or which are not considered to be representative of conventional municipal sewage lagoons. The remaining 137 facilities fall into five basic categories:

- (i) continuous discharge facultative lagoons
- (ii) annual discharge facultative lagoons
- (iii) seasonal discharge facultative lagoons
- (iv) aerated lagoons
- (v) combined aerated cells and facultative lagoons (aerobic - facultative lagoons)

A listing of the conventional lagoons in each category is provided in Appendix 6. For costing purposes, the 10-percentile, 50-percentile and 90-percentile sized lagoon facility was used, based upon the distribution in lagoon design capacities which exist throughout Ontario. This distribution is as follows:

- 10-percentile - 300 m^3/d
- 50-percentile - 1000 m^3/d
- 90-percentile - 3300 m^3/d

Section 4.1.1 describes this issue in more detail.

Each lagoon category is described in more detail below.

TABLE 2.1 [1]

CLASSIFICATION OF LAGOONS IN ONTARIO IN 1990

Type	Total (1990)	North[8]			South[9]		
		Total	Without Phosphorus Removal	With Phosphorus Removal	Total	Without Phosphorus Removal	With Phosphorus Removal
Continuous Discharge	11	3[1]	2	1	8[2]	6	2
Annual Discharge	14	1	0	1	13	8	5
Seasonal Discharge	92	36[6]	20	16	56[7]	14	42
Aerated Lagoons	5	2	2	0	3[3]	1	2
Aerobic - Facultative Lagoons	15	4[4]	2	2	11[5]	3	8
TOTAL	137	46	26	20	91	32	59

Notes:
1. Excludes Val Gagne (summer storage).
2. Excludes Chesley & Sutton, Beaverton River (exfiltration) and Lindsay (aerated cell).
3. Excludes Colchester South (operational April 89), Markdale (exfiltration), and Exeter (summer storage).
4. Includes North Goward (aerated cell & continuous discharge lagoon).
5. Excludes Listowel (spray), New Hamburg & Stayner, Tobermory (exfiltration) and Aylmer, Dundalk and Thornbury (summer storage). Includes Milverton.
6. Excludes Garson (standby).
7. Excludes Milverton (aerated cell), Dutton & Rodney, Bracebridge (aerated cell and polishing) and Cainsville (industrial).
8. 'North' is defined as MOE Northeastern and Northwestern Regions.
9. 'South' is defined as MOE Southeastern, West Central, Central and Southwestern Regions.

2.2 Continuous Discharge Facultative Lagoons

2.2.1 Process Description

Facultative ponds are generally 1.2 to 2.5 m in depth with an aerobic layer overlying an anaerobic layer, often containing sludge deposits (U.S.EPA, 1983). Continuous discharge facultative ponds, as the name implies, continuously receive raw sewage and discharge treated effluent year-round to the receiving water. These systems may be operated with or without phosphorus removal capability. In those facilities which are required to remove phosphorus, a chemical precipitant, usually alum, is added on a continuous basis to the raw sewage influent to the lagoon. The chemical may be added at a pumping station to facilitate mixing or to a turbulent area at the lagoon influent. In some continuous discharge lagoons, chemicals are also added on a batchwise basis to the lagoon contents periodically to improve phosphorus removal. The chemical precipitant reacts with the soluble phosphorus present in the raw wastewater and the precipitated metal phosphate settles along with other particulate matter in the lagoon.

Within the lagoon, anaerobic degradation of organic matter occurs in the lower layers which are devoid of oxygen. Aerobic stabilization occurs in the upper layers during ice-free periods when photosynthetic algae and surface re-aeration provide oxygen which can be utilized by the aerobic bacteria.

As identified in Table 2.1, there were a total of eleven continuous discharge lagoons in Ontario in 1990 based on the UMIS database. Of these, three provided phosphorus removal and eight did not.

2.2.2 Design Guidelines

MOE design guidelines (MOE, 1984) suggest the following key process design considerations for continuous discharge waste stabilization ponds:

(i) maximum operating depth 1.8 m

(ii) loading should not exceed 22 kg BOD_5/ha.d for significant periods

(iii) minimum of two cells, with maximum of 8 ha per cell and 4 ha per cell preferred

(iv) piping interconnection between cells to permit flow between cells as well as series or parallel operation

(v) ability to introduce raw sewage to any cell is desirable

(vi) effluent piping as far removed as possible from inlet and cross-connection piping

2.2.3 Performance Characteristics

The effluent quality which a well-operated continuous discharge lagoon treating normal strength wastewater can be expected to produce is presented in Table 2.2. (MOE, 1984). Effluent quality requirements for those facilities which do not have specific effluent limits embodied in a Certificate of Approval are based on Policy 08-01 and the Canada-U.S. Agreement. These limits are also shown in Table 2.2.

The eleven continuous discharge lagoons were subdivided into four categories by location (Northern and Southern Ontario) and phosphorus removal capability. Based on UMIS data for the period 1986 through 1990, the average effluent quality for each of the four sub-categories of continuous discharge lagoons are presented in Table 2.3. It should be noted that there were few examples of lagoons in each of the sub-categories. Therefore, the averages calculated are based on very few facilities.

For subcategories of lagoons which contained five or more examples, five year (1986 - 1990) seasonal effluent quality averages were also calculated. This only applied to continuous discharge lagoons in southern Ontario which were not practising phosphorus removal. The seasonal averages for this category of lagoon are included in Table 2.3. Effluent BOD_5 and phosphorus concentrations in effluents from these facilities did not show any significant amount of seasonal variations. Effluent TSS concentrations were higher in summer and fall periods than in winter or spring, reflecting the increase in algal

TABLE 2.2

DESIGN OBJECTIVES AND EFFLUENT QUALITY OBJECTIVES FOR CONTINUOUS DISCHARGE LAGOONS

	Concentration (mg/L)		
	BOD_5 (Annual Average)	**Suspended Solids (Annual Average)**	**Total Phosphorus (Monthly Average)**
Expected Effluent Quality*			
With P Removal	25	30	1.0
Without P Removal	25	30	6.0
General Effluent Quality Requirements**			
With P Removal	30	40	1.0
Without P Removal	30	40	n/a

n/a = not applicable

* According to MOE Guidelines for the Design of Sewage Treatment Works (MOE,1984).

** Based on MOE Policy 08-01 and the Canada - U.S. Agreement on Great Lakes Water Quality, October 1983.

TABLE 2.3

EFFLUENT QUALITY ACHIEVED BY CONTINUOUS DISCHARGE LAGOONS IN ONTARIO*

Geographical Location	Phosphorus Removal Capability	Number of Facilities	Basis of Average	Effluent Quality (Avg ±Std. Dev, mg/L)			
				BOD_5	TSS	TP	NH_3-N
South	Yes	2	Annual	12.0 ±3.2	13.2 ±0.0	0.48 ±0.0	4.4 ±2.0
South	No	6	Annual	15.6 ±7.5	26.4 ±15.9	2.31 ±1.03	4.0 ±1.5
			Spring	17.3 ±6.8	24.0 ± 8.8	2.25 ±0.97	6.6 ±1.8
			Summer	16.5 ±8.1	31.5 ±28.1	2.23 ±0.89	2.7 ±1.6
			Fall	13.5 ±7.2	28.6 ±19.4	2.10 ±1.18	2.3 ±1.3
			Winter	13.8 ±7.1	22.8 ±11.3	2.61 ±1.35	6.6 ±3.0
North	Yes	1	Annual	8.2	14.1	1.18	5.3
North	No	2	Annual	31.5 ±0.5	33.2 ±7.3	4.92 ±1.37	20.0 ±10.0

* Based on performance for 1986 to 1990.

growth in these periods. Ammonia concentrations were lower in summer and fall due to increased photosynthetic activity in these periods.

In order to establish the effluent limits that continuous discharge lagoons are capable of achieving on a monthly average basis, a statistical analysis of actual lagoon performance data was conducted using the procedures outlined in the MISA Issues Resolution Process final report (MOE, 1991). A brief description of the statistical analysis procedures is included in Appendix 5. This procedure was only applied to those sub-categories of lagoon type which had five or more individual facilities. The results of this analysis are presented in Table 2.4.

Limited toxicity data were available for effluents from two continuous discharge lagoons at Perth and Lindsay (prior to the conversion of the Lindsay plant to a Sutton Process). At these two plants, 24-hour composite and grab final effluent samples were collected under both summer and winter operating conditions. Testing included 96-hour rainbow trout acute lethality tests, 48-hour Daphnia magna acute lethality tests, fathead minnow (Pimephales promelas) larval survival and growth tests and Ceriodaphnia dubia chronic toxicity tests. The results of this testing are summarized in Table 2.5 (Beak, 1991).

Rainbow trout exhibited more sensitivity to the conventional lagoon effluents than Daphnia magna. Winter discharges (February and March) exhibited more lethality to trout than summer discharges (August and September). Summer discharges were generally not acutely toxic. Both winter and summer samples displayed some chronic toxicity effects, but summer samples overall had lower chronic values than winter samples.

2.2.4 Costs of Treatment

Treatment costs for continuous discharge facultative lagoons have been compiled both for the capital cost of the facility installation as well as the expected yearly operations and maintenance costs. These have been indicated in the capital and operations cost curves in Figures 2.1 and 2.2 respectively. Specific costing assumptions have been included in

TABLE 2.4 PERFORMANCE CAPABILITIES OF CONTINUOUS DISCHARGE LAGOONS						
Geographical Location	**Phosphorus Removal Capability**	**Number of Facilities**	**Effluent Limit* (Monthly Average)**			
			BOD_5	**TSS**	**TP**	**NH_3-N**
South	Yes	2	n/a	n/a	n/a	n/a
	No	6	30	40	n/a	10.0
North	Yes	1	n/a	n/a	n/a	n/a
	No	2	n/a	n/a	n/a	n/a

n/a = not applicable

* Limits were not calculated for any category containing fewer than five facilities.

TABLE 2.5

SUMMARY OF CONTINUOUS DISCHARGE LAGOON EFFLUENT TOXICITY DATA (BEAK, 1991)

				Rainbow Trout	Daphnia magna	Fathead Minnow		Ceriodaphnia	
STP	Season	Date Sample Collected (dd.mm.yy)	Sample Type	LC50 (%v/v)	LC50 (%v/v)	Chronic Value (%v/v)	LC50 (%v/v)	Chronic Value (%v/v)	LC50 (%v/v)
Perth	Summer	13.09.89	C	NL	NL	84	NL	>100	NL
		14.09.89	C	NL	NL	84	NL	>100	NL
		15.09.89	C	NL	NL	39	NL	>100	NL
		16.09.89	C	NL	NL	59	NL	>100	NL
		13.09.89	G	NL	NL				
		14.09.89	G	NL	NL				
		15.09.89	G	NL	NL				
	Winter	21.02.90	C	54	NL	39	NL	59	NL
		22.02.90	C	98	NL	59	NL	24	31
		23.02.90	C	75	NL	39	NL	39	NL
		24.02.90	C	78	NL	39	NL	39	24
		21.02.90	G	59	NL				
		22.02.90	G	57	NL				
		23.02.90	G	18	NL				
Lindsay	Summer	23.08.89	C	NL	NL	>100	NL	>100	NL
		24.08.89	C	NL	NL	24	NL	>100	NL
		25.08.89	C	NL	NL	>100	NL	>100	NL
		26.08.89	C	NL	NL	>100	NL	>100	NL
		23.08.89	G	>100	NL				
		24.08.89	G	NL	NL				
		25.08.89	G	NL	NL				
	Winter	29.03.89	C	96	NL	71	67	71	NL
		30.03.89	C	63	NL	71	70	14	NL
		31.03.89	C	71	NL	39	100	71	75
		01.04.89	C	71	>100	39	93	>100	NL
		29.03.89	G	67	NL				
		30.03.89	G	100	NL				
		31.03.89	G	71	NL				

Code : C = Composite Sample G = Grab Sample NL = Non-Lethal

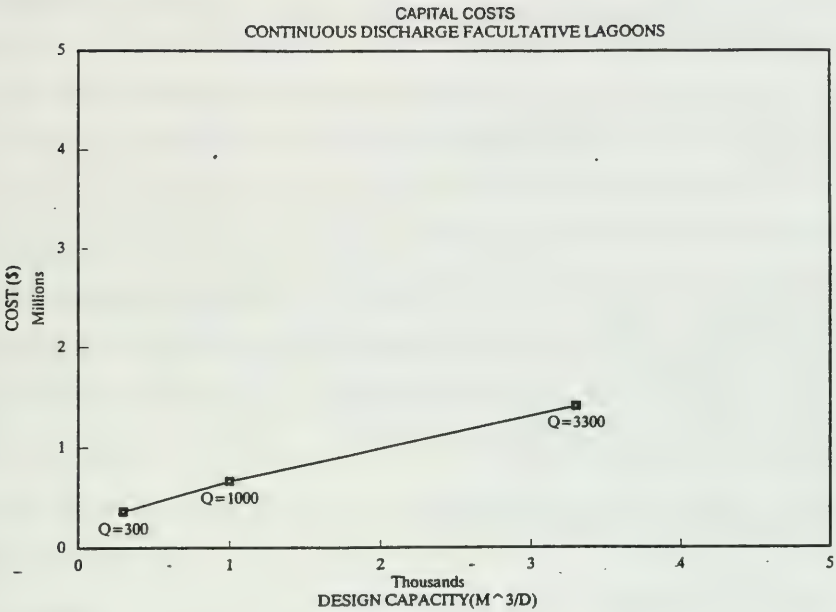


FIGURE 2.1

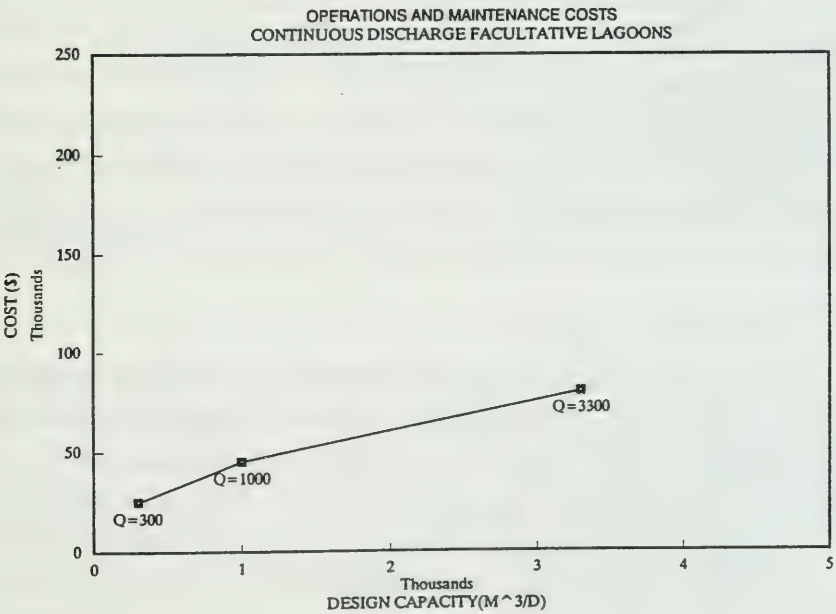


FIGURE 2.2

Appendix 4. All costs are in 1992 dollars using a March /92 ENR Index = 6537.

As indicated in Figure 2.1, the range in capital cost between the 10 and 90-percentile plant sizes is \$360,000 - \$1,400,000, translating to \$1200/m^3/d (\$5,500,000/MGD) for the 300 m^3/d (\$2,000,000/MGD) sized plants to \$430/m^3/d for the 3300 m^3/d sized plants. The costing has been done on the basis of providing for phosphorus removal through alum addition. Yearly amortized capital costs plus operating and maintenance costs range from \$206/m^3.d (\$940,000/MGD) to \$68/m^3.d for the 300 m^3/d to 3300 m^3/d sizes, while the percent capital cost contribution ranges from 59% to 64% over this same size range (See Appendix 4 for cost estimate breakdowns). Amortized costs are based upon a 20 year design life, using an interest rate of 8%.

2.3 Fill-and-Draw Facultative Lagoons

2.3.1 <u>Process Description</u>

Receiving water constraints often require that lagoons be operated in the fill-and-draw mode in which discharge from the lagoon is only allowed during specific periods of the year. Typically, fill-and-draw lagoons are discharged once per year in the fall (annual discharge facultative lagoons) or twice per year in the spring and fall (seasonal lagoons). Spring discharge usually occurs after the ice cover leaves the lagoon to reduce hydrogen sulphide and ammonia concentrations in the effluent. The discharge rate may also need to be controlled in proportion to receiving stream flow to provide adequate dilution of the lagoon effluent.

The treatment processes occurring in fill-and-draw lagoons are the same as those that occur in continuous discharge lagoons. Phosphorus removal, if required, is achieved by addition of a chemical precipitant, usually alum. The chemical can be added continuously to the lagoon influent or the lagoon contents can be treated batch-wise just prior to discharge. Continuous chemical addition has not proven to be as effective as batch treatment in accomplishing phosphorus removal (MOE, 1984).

As identified in Table 2.1, the vast majority (77%) of the facultative lagoons in Ontario operate in the fill-and-draw mode. Of these, the majority (87%) are seasonal discharge lagoons. Annual discharge lagoons are more common in the southern part of the province than in the north where there is only one example of this type of facility.

2.3.2 Design Guidelines

Design guidelines for fill-draw lagoons are similar to those for continuous discharge lagoons with the exception of the additional holding time which must be provided in the fill-and-draw systems (MOE, 1984). The discharge system must be designed to accommodate the required discharge rate and capability to proportion the discharge to the stream flow may be required. The bottom 0.3 m content of the lagoon must be retained following discharge to prevent the discharge of accumulated solids to the receiving stream.

2.3.3 Performance Characteristics

The effluent quality which a well-operated fill-and-draw lagoon treating normal strength wastewater can be expected to produce is presented in Table 2.6. Lagoons which batch treat for phosphorus removal are expected to achieve better effluent quality than either continuous discharge lagoons or fill-and-draw lagoons which add chemicals continuously to accomplish phosphorus removal. This is also reflected in the effluent quality requirements of Policy 08-01 as summarized in Table 2.6.

The fill-and-draw lagoons were divided into two categories by discharge type - annual discharge and seasonal discharge. These categories were then further subdivided by geographical location and phosphorus removal capability. Average effluent quality for each subcategory of plant was then calculated based on performance data for the period 1986 through 1990. These data are presented in Table 2.7 (annual discharge) and Table 2.8 (seasonal discharge). Average qualities of spring and fall discharges were also calculated for the seasonal discharge lagoons and are included in Table 2.8.

TABLE 2.6

DESIGN OBJECTIVES AND EFFLUENT QUALITY OBJECTIVES FOR FILL-AND-DRAW LAGOONS

	Concentration (mg/L)		
	BOD (Annual Average)	**Suspended Solids (Annual Average)**	**Total Phosphorus (Monthly Average)**
Expected Effluent Quality*			
Without P Removal	25	30	6.0
Batchwise P Removal	15	20	1.0/0.5
Continuous P Removal	25	30	1.0
General Effluent Quality Requirements**			
Without P Removal	30	40	n/a
Batchwise P Removal	25	25	1.0
Continuous P Removal	30	40	1.0

n/a = not applicable

* According to MOE Guidelines for the Design of Sewage Treatment Works (MOE,1984)

** Based on MOE Policy 08-01 and the Canada - US Agreement on Great Lakes Water Quality, October, 1984

TABLE 2.7

EFFLUENT QUALITY ACHIEVED
BY ANNUAL DISCHARGE LAGOONS
IN ONTARIO*

Geographical Location	Phosphorus Removal Capacity	Number of Facilities	Effluent Quality (Avg ± Std. Dev., mg/L)			
			BOD_5	TSS	TP	NH_3-N
South	Yes	5	6.8 ± 1.5	35.7 ± 22.5	0.55 ± 0.20	1.4 ± 0.6
	No	8	9.4 ± 6.3	36.4 ± 25.4	2.34 ± 1.69	2.7 ± 4.3
North	Yes	1	ND	ND	ND	ND
	No	0	n/a	n/a	n/a	n/a

n/a = not applicable

ND = No Data

* Based on performance for 1986 to 1990

TABLE 2.8

EFFLUENT QUALITY ACHIEVED
BY SEASONAL DISCHARGE LAGOONS
IN ONTARIO*

Geographical Location	Phosphorus Removal Capability	Number of Facilities	Basis of Average	Effluent Quality (Avg ±Std. Dev., mg/L)			
				BOD_5	TSS	TP	NH_3-N
South	Yes	42	Overall	10.7 ±4.5	28.8 ±17.5	0.55 ±0.23	3.4 ±2.2
			Spring	12.4 ±8.6	27.3 ±32.7	0.62 ±0.46	4.7 ±4.8
			Fall	8.1 ±5.4	21.4 ±26.5	0.49 ±0.33	2.5 ±2.7
South	No	14	Overall	12.5 ±4.9	26.9 ±18.7	1.76 ±0.97	6.2 ±2.8
			Spring	14.6 ±9.2	29.2 ±25.1	1.98 ±1.32	8.8 ±10.3
			Fall	8.6 ±6.6	20.9 ±28.3	1.74 ±1.33	4.0 ±3.1
North	Yes	16	Overall	6.2 ±4.3	20.1 ±15.0	0.70 ±0.33	4.8 ±7.4
			Spring	8.8 ±6.9	17.8 ±15.9	0.91 ±0.61	7.3 ±11.5
			Fall	5.0 ±4.0	14.3 ±22.8	0.53 ±0.36	3.1 ±2.8
North	No	20	Overall	11.1 ±5.3	28.7 ±26.2	2.27 ±1.78	4.6 ±3.2
			Spring	16.8 ±12.7	33.3 ±36.1	3.53 ±4.67	8.8 ±9.3
			Fall	7.8 ±7.9	23.4 ±82.9	2.21 ±1.70	4.8 ±4.4

* Based on performance data for 1986 to 1990

Using the data analysis procedures outlined in the MISA Issues Resolution Process final report (MOE, 1991, refer to Appendix 5), performance limits were established for each subcategory of fill-and-draw facultative lagoon which had five or more examples. This included the annual discharge lagoons in Southern Ontario and all categories of seasonal discharge lagoons. These limits are reported in Table 2.9. For the seasonal discharge lagoons, performance limits were developed separately for spring discharges and fall discharges. The effluent quality capabilities of all types of seasonal discharge lagoons were poorer in terms of ammonia and TSS in the spring than in the fall. This reflects the effect of ice cover in the winter months. TSS concentrations were generally highest in spring discharges, possibly due to the turnover of the lagoon contents in the spring months. Differences in terms of other parameters was minimal.

2.3.4 Costs of Treatment

The costs of treatment for the range of sizes of fill and draw facultative lagoons has been indicated by the capital and operations cost curves of Figures 2-3 to 2-6. This includes curves for both seasonal and annual discharge lagoons. Seasonal discharge lagoons range in cost between the 10 and 90-percentile design size of $600,000 to $3,350,000, while annual discharge lagoons range between $800,000 to $5,000,000. This difference in cost is primarily attributable to lagoon size, with seasonal lagoons providing for 180 days storage, while annual lagoons provide for 360 days storage. Operation and maintenance costs are virtually identical.

For the seasonal discharge lagoons, yearly amortized capital plus operation and maintenance costs range from \$279/m^3.d (\$1,270,000/MGD) to \$125/m^3.d (\$570,000/MGD) for the 300 - 3300 m^3/d sizes (74% to 82% range in capital cost contribution). For annual discharge lagoons, this yearly amortized capital plus operation and maintenance cost ranges from \$344/m^3.d (\$1,560,000/MGD) to \$175/m^3.d (\$780,000/MGD) over the same size ranges (79% to 87% range in capital cost contribution).

TABLE 2.9

PERFORMANCE CAPABILITIES
OF FILL-AND-DRAW LAGOONS

Lagoon Type	Geographical Location	Phosphorus Removal Capability	Number of Facilities	Basis of Limit	Effluent Limit (mg/L)			
					BOD_5	TSS	TP	NH_3-N
Annual	South	Yes	5	n/a	15	90	1.5	4.0
	South	No	8	n/a	15	65	n/a	4.0
	North	Yes	1	n/a	n/a	n/a	n/a	n/a
	North	No	0	n/a	n/a	n/a	n/a	n/a
Seasonal	South	Yes	42	Overall	20	45	1.0	7.0
				Spring	20	40	1.0	8.0
				Fall	15	30	1.0	6.0
	South	No	14	Overall	20	40	n/a	14.0
				Spring	20	45	n/a	14.0
				Fall	30	30	n/a	10.0
	North	Yes	16	Overall	15	35	1.5	7.0
				Spring	15	25	1.5	10.0
				Fall	10	30	1.0	6.0
	North	No	20	Overall	20	45	n/a	7.0
				Spring	20	50	n/a	10.0
				Fall	15	25	n/a	7.0

n/a = not applicable

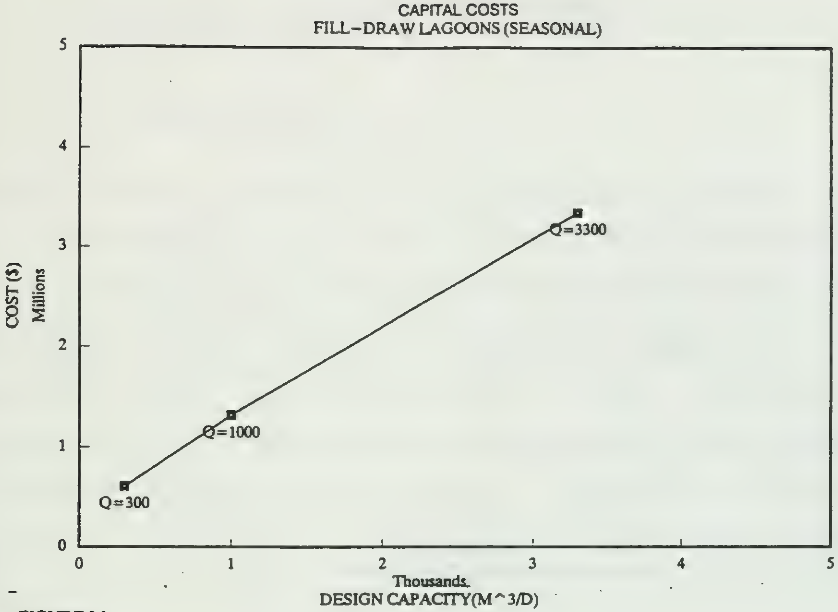


FIGURE 2.3

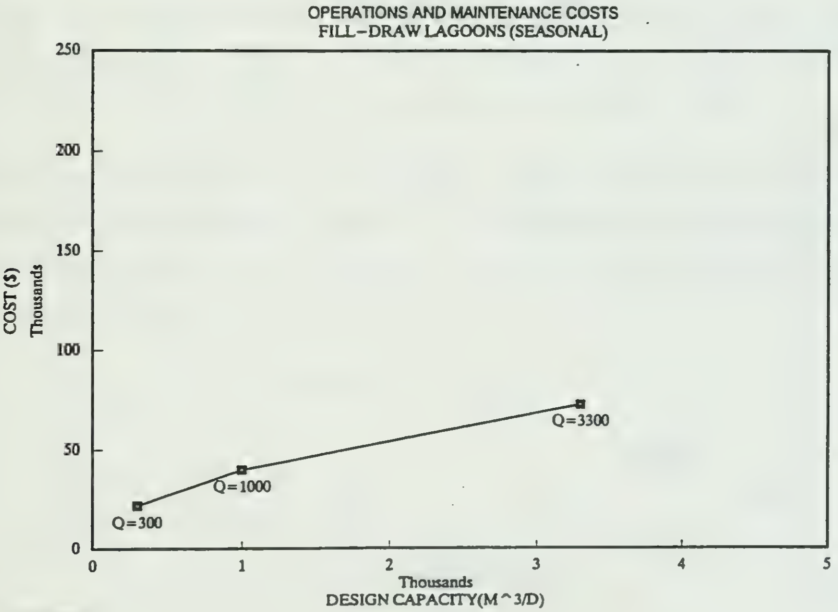


FIGURE 2.4

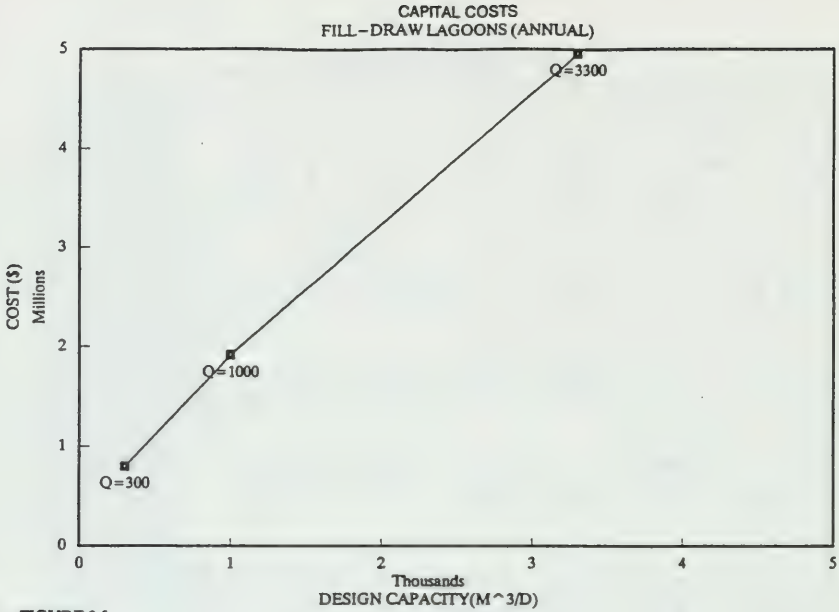


FIGURE 2.5

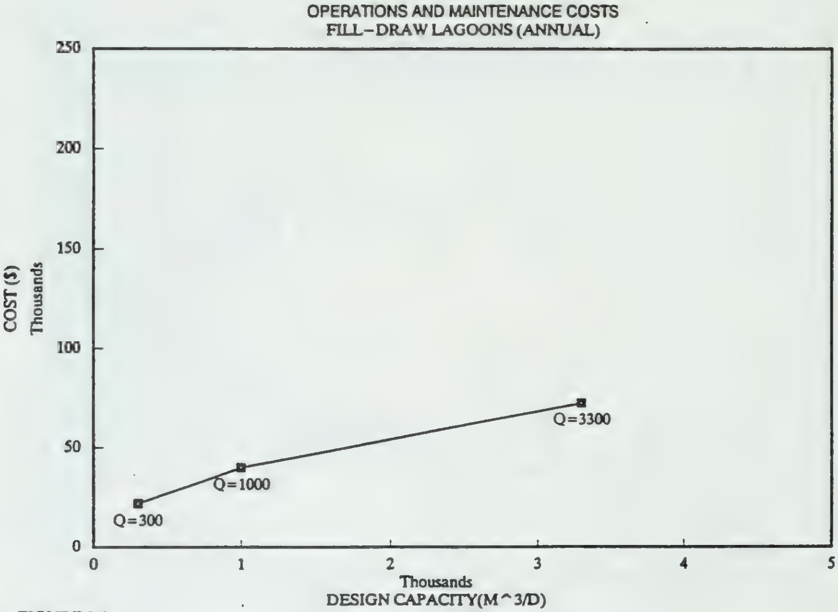


FIGURE 2.6

2.4 Aerated Lagoons

2.4.1 Process Description

Organic loading rates on facultative lagoons can be increased if supplemental oxygen is provided by mechanical means to augment the oxygen available due to photosynthetic activity and surface re-aeration. The supplemental oxygen can be supplied by mechanical surface aerators or diffused aeration. Submerged aeration systems are recommended where extreme winter temperatures are experienced (MOE, 1984). Adequate oxygen is provided to maintain aerobic conditions throughout the liquid phase. However, the energy input is low enough to allow suspended particulate matter to settle to the bottom of the lagoon for subsequent anaerobic degradation. Because of the mechanical input of oxygen to aerated lagoons, these systems operate at higher loadings and can be deeper than facultative lagoons. Hence, they require considerably less land space than facultative ponds of the same capacity (U.S.EPA, 1983).

As with other lagoon systems, aerated lagoons can be operated with or without phosphorus removal. Since these lagoons are generally continuously discharged, chemicals are normally added on a continuous basis to the lagoon influent.

There were only five aerated lagoons operating in Ontario in 1990 with continuous direct discharge to the receiving stream (Table 2.10). These were equally distributed over the northern and southern part of the province. Two of the five aerated lagoons practised phosphorus removal.

2.4.2 Design Guidelines

MOE design guidelines suggest that aerated lagoons which provide total retention time of approximately 30 days including the time in a quiescent zone to permit settling should be capable of producing an effluent quality equivalent to that produced by a conventional activated sludge plant (MOE, 1984). No guidelines for organic loading are suggested. U.S. EPA suggests detention times of 3 to 10 days for aerated lagoons (U.S. EPA, 1983). Basin mixing requirements in the aerated cell in the range of 1.6 to 2.5 W/m^3 are recommended to maintain uniform dissolved oxygen concentrations (MOE, 1984).

2.4.3 Performance Characteristics

Typical effluent quality characteristics for aerated lagoons alone are not indicated by MOE (MOE, 1984). These systems are subject to the same effluent quality requirements as other continuous discharge lagoons, namely 30 mg/L BOD_5 and 40 mg/L TSS on an annual average basis and 1 mg/L TP on a monthly average basis if required by the Canada-U.S Agreement.

Since there were no more than two examples of aerated lagoons in any geographical or phosphorus removal category, available performance data for these facilities is very limited. Average effluent quality data for these plants for the period 1986 through 1990 are summarized in Table 2.10. Since no category of aerated lagoon contained five or more plants, no seasonal analysis of discharge quality or analysis of performance capabilities was undertaken.

2.4.4 Costs of Treatment

Costs of treatment of aerated lagoons is based upon using submerged (diffused) aeration systems with blowers housed in a central control building. These costs have been indicated in Figure 2.7 and 2.8. As can be seen, the capital costs of these facilities is less than that indicated previously with the seasonal fill-draw lagoons, due to the decreased

TABLE 2.10

EFFLUENT QUALITY ACHIEVED BY AERATED LAGOONS IN ONTARIO*

Geographical Location	Phosphorus Removal Capability	Number of Facilities	Effluent Quality (Avg ±Std. Dev., mg/L)			
			BOD_5	TSS	TP	NH_3-N
South	Yes	2	9.6 ±1.8	12.1 ±0.1	0.72 ±0.08	5.5 ±2.3
South	No	1	10.2	11.7	1.25	5.5
North	Yes	0	n/a	n/a	n/a	n/a
North	No	2	15.6 ±0.9	18.2 ±3.1	2.29 ±0.80	8.0 ±0.0

n/a = not applicable

* Based on performance data for 1986 to 1990

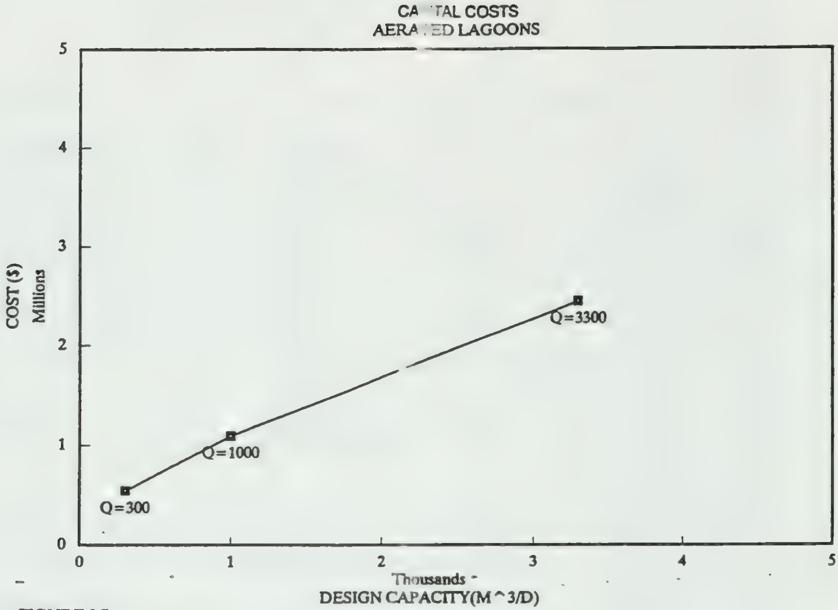


FIGURE 2.7

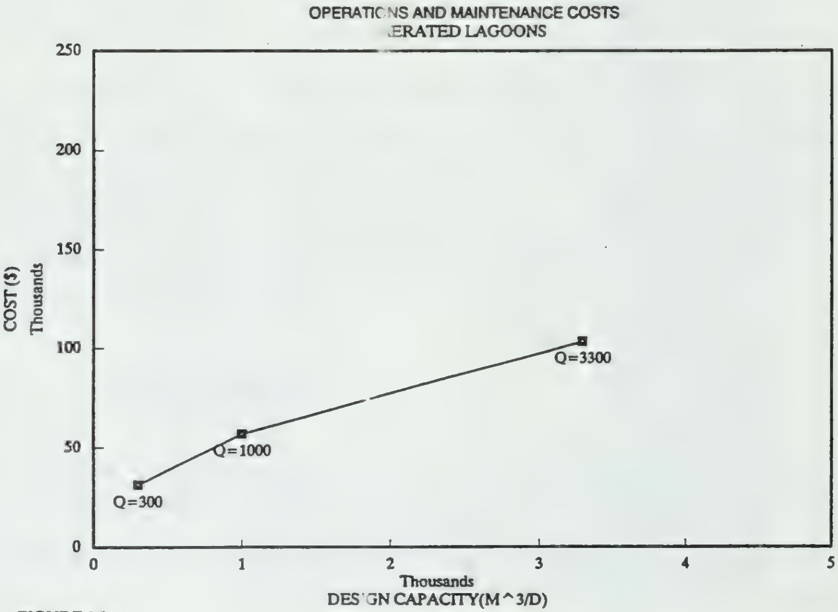


FIGURE 2.8

lagoon size required. Operations costs are, however, higher than facultative lagoons due to the power requirements of the aeration system.

Yearly amortized capital plus operating and maintenance costs range from $287/m^3.d ($1,300,000/MGD) to $108/m^3.d ($490,000 MGD) for the 300 m^3/d to 3300 m^3/d sizes. The percent capital cost contribution over this same size range is 64% to 71%.

2.5 Aerobic - Facultative Lagoons

2.5.1 Process Description

Aerobic lagoons, as described in Section 2.4, are most commonly used for pretreatment upstream of a facultative pond. This process configuration is termed an aerobic - facultative lagoon. The majority of the organic degradation occurs aerobically in the aerated pond. The effluent from the aerobic lagoon with about 4 to 5 days retention time will contain about 60 mg/L BOD_5 and 100 mg/L TSS. Polishing and settling occurs in the facultative pond. In this process configuration, the facultative pond can be about one-third as large as would be required without the aerated pond because the aerated pond removes about two-thirds of the raw sewage BOD_5.

Phosphorus removal in aerobic - facultative lagoons can be accomplished by continuous chemical addition to either the aerated lagoon or the facultative lagoon.

There were a total of 15 aerobic - facultative lagoons in Ontario in 1990 of which 11 were located in southern part of the province (Table 2.1). Two-thirds of these lagoon systems were operated with chemical addition to achieve phosphorus removal.

2.5.2 Design Guidelines

MOE recommends design organic loadings for the aerated cell of an aerobic - facultative pond system in the range from 0.031 to 0.048 kg/m^3.d, if the system is not required to

nitrify (MOE, 1984). Loadings for the facultative pond would be the same as those identified in Section 2.2.2 to achieve the same design effluent quality as a continuous discharge facultative pond. Basin mixing requirements in the aerated cell in the range of 1.6 to 2.5 W/m^3 are recommended to maintain uniform dissolved oxygen concentrations (MOE, 1984). Four to five days of hydraulic retention time are recommended in the aerated cell to achieve the degree of pretreatment suggested in Section 2.5.1.

2.5.3 Performance Characteristics

Typical effluent quality characteristics and effluent quality requirements for aerobic - facultative lagoons in Ontario are the same as applied to facultative lagoons.

Average performance characteristics for aerobic - facultative lagoons are summarized in Table 2.11 for each geographical area and phosphorus removal capability. Seasonal variations in effluent quality were only calculated for southern Ontario aerobic - facultative lagoons with phosphorus removal capability since this category of plant was the only one which had a representative number ($\geq$ 5) of facilities. Similarly, performance capabilities based on the MISA calculation procedure (MOE, 1991; refer to Appendix 5) were only calculated for southern Ontario systems with phosphorus removal capability. These monthly average performance limits were calculated based on year-round performance data as well as for spring and fall periods. These data are summarized in Table 2.12.

2.5.4 Costs of Treatment

Treatment costs for the aerated-facultative lagoon system range from $400,000 for the 10-percentile sized plant to $1,700,000 for the 90-percentile sized plant (see Figures 2.9 and 2.10). This is less than that indicated previously in Section 2.4.4. for aerated lagoons, primarily due to the fact that the aeration system is not as large, since facultative lagoons are used for further polishing and settling in the final step of the process.

TABLE 2.11

EFFLUENT QUALITY ACHIEVED BY AEROBIC - FACULTATIVE LAGOONS IN ONTARIO*

Geographical Location	Phosphorus Removal Capability	Number of Facilities	Basis of Average	Effluent Quality (Avg ±Std. Dev., mg/L)			
				BOD_5	TSS	TP	NH_3-N
South	Yes	8	Annual	14.8 ±8.0	25.4 ±12.7	0.81 ±0.43	6.4 ±2.7
			Spring	18.5 ±6.0	26.9 ±9.9	0.82 ±0.51	6.7 ±3.6
			Summer	21.5 ±19.6	28.5 ±20.4	0.86 ±0.79	4.3 ±2.8
			Fall	11.1 ±7.6	19.5 ±12.6	0.57 ±0.36	4.7 ±3.0
			Winter	19.0 ±16.4	21.4 ±13.9	0.81 ±0.48	11.3 ±9.5
South	No	3	Annual	17.9 ±6.5	20.2 ±6.4	2.84 ±0.56	7.1 ±4.1
North	Yes	2	Annual	12.3 ±4.2	14.2 ±8.4	1.96 ±0.80	7.0 ±0.7
North	No	2	Annual	14.4 ±0.1	16.5 ±0.9	1.75 ±0.13	4.8 ±0.4

* Based on performance data for 1986 to 1990

TABLE 2.12

PERFORMANCE CAPABILITIES OF AEROBIC - FACULTATIVE LAGOONS

Geographical Location	Phosphorus Removal Capability	Number of Facilities	Basis of Limit	Effluent Limit (mg/L)			
				BOD_5	TSS	TP	NH_3-N
South	Yes	8	Overall	25	45	1.5	12
			Spring	25	50	1.5	16
			Fall	20	30	1.5	9
South	No	3	n/a	n/a	n/a	n/a	n/a
North	Yes	2	n/a	n/a	n/a	n/a	n/a
North	Yes	2	n/a	n/a	n/a	n/a	n/a

n/a = not applicable

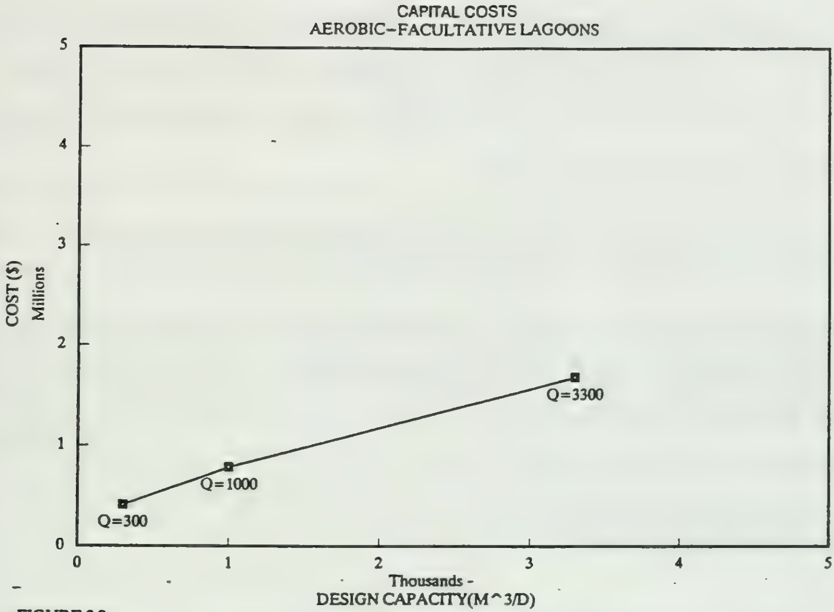


FIGURE 2.9

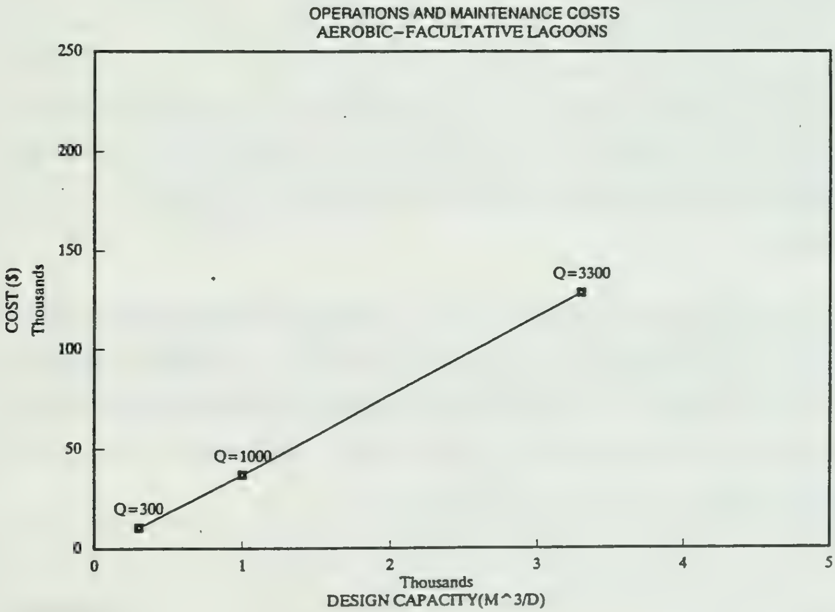


FIGURE 2.10

The yearly amortized capital plus operating and maintenance costs for aerobic - facultative lagoons range from \$173/m^3.d (\$790,000/MGD) to \$91/m^3.d (\$410,000/MGD) for the 300 m^3/d to 3300 m^3/d sizes, while the percent capital cost contribution ranges from 80% to 57% over this same size range.

2.6 Summary of Treatment Capabilities of Conventional Lagoon Systems

Facultative lagoons are effective low cost wastewater treatment processes which can, under ideal circumstances, produce an effluent quality equivalent to that achievable by conventional secondary treatment facilities. However, these facilities are subject to significant seasonal effects due to climatic conditions as evident from the data presented in Sections 2.2 through 2.5 and from information available from other sources (U.S. EPA, 1983; Environment Canada, 1985).

Winter and spring discharges from continuous discharge lagoons have higher concentrations of ammonia as demonstrated by data in Table 2.3. The toxicity of samples collected in winter and early spring from continuous discharge lagoons was higher than that of samples collected in summer and fall periods (Beak, 1991). This is likely related to elevated ammonia and H_2S concentrations in the effluent from these facilities under ice-covered conditions. The elevated pH of lagoon effluents in summer and fall may also affect effluent toxicity since a greater fraction of the ammonia will be present in the more toxic unionized form.

Spring and fall discharges from annual discharge and seasonal discharge lagoons can have elevated TSS concentrations due to spring turnover and the presence of algal cells (Table 2.7). Many categories of lagoons will have difficulty achieving monthly average phosphorus limits of 1.0 mg/L based on monthly effluent quality capabilities calculated from available performance data (Table 2.9 and 2.12).

3.0 ALTERNATIVES FOR UPGRADING LAGOON EFFLUENT QUALITY

3.1 General

As noted in Section 2.6, conventional lagoons are subject to seasonal variations in effluent quality. Elevated concentrations of suspended solids may result in non-compliance with effluent limits which are based on monthly average values. Elevated ammonia and H_2S concentrations may result in toxic effluents in the spring. Hence, to determine BATEA effluent limits for small municipalities which historically rely upon lagoon systems for sewage treatment, cost effective methods for upgrading lagoon effluent quality need to be identified.

Two processes have been applied in Ontario for upgrading lagoon effluents to meet more stringent TSS and ammonia limits. These are:

- i) the "Sutton Process"; and,
- ii) the intermittent sand filtration process or "New Hamburg Process".

The Sutton Process has been implemented at eight Ontario locations since it was first demonstrated at Sutton, Ontario starting in 1981. The facilities currently operating in the Sutton Process operating mode are identified in Table 3.1, along with their design capacity and the date of start-up.

The intermittent sand filtration process is currently operating at two locations in Ontario as identified in Table 3.2. The original application of this technology was at New Hamburg, Ontario; hence, the process has been called the "New Hamburg Process" in Ontario.

Detailed discussions of the Sutton Process and the New Hamburg Process are presented in Sections 3.2 and 3.3, respectively. In addition, other approaches which have not been widely applied in Ontario but which have been used elsewhere in North America are

Table 3.1 Examples of Sutton Process Plants		
Facility	**Design Capacity (m^3/d)**	**Start-Up Date**
Sutton WPCP	2,046	1981
Colborne WPCP	1,375	1983
Cookstown WPCP	825	1988
Tottenham WPCP	2,257	1988
Lindsay WPCP	15,870*	1990
Dutton WPCP	558	1991
Rodney WPCP	590	1991
Stayner WPCP	1,875	1991

*Flow limited to 12,672 m^3/d in C of A.

Table 3.2 Examples of New Hamburg Process Plants		
Facility	**Design Capacity (m^3/d)**	**Start-Up Date**
New Hamburg WPCP	2,700*	1981
Schomberg WPCP	683	1991

*Flow limited to 2,300 m^3/d in C of A.

described briefly in Section 3.4. More detailed descriptions of the design and operation of each of the eight existing Sutton Process plants are included in Appendix 1. Detailed descriptions of the two New Hamburg Process plants are included in Appendix 2. Appendix 3 contains a more extensive review of other approaches for upgrading lagoon effluent quality.

3.2 The Sutton Process

3.2.1 Process Description

In the Sutton Process, raw sewage undergoes simultaneous carbon oxidation and nitrification in a single sludge extended aeration activated sludge plant prior to polishing in a facultative waste stabilization pond. The process is shown schematically in Figure 3.1. Long sludge ages (SRT) and high mixed liquor suspended solids concentrations are maintained in the extended aeration plant to ensure year-round nitrification. The nitrified effluent from the extended aeration plant contains sufficient concentration of nitrates to ensure that anoxic conditions are maintained in the pond despite ice cover in the winter months. Thus, anaerobic conditions which contribute to H_2S formation are avoided. The unique feature of the Sutton Process is that excess biological solids produced in the extended aeration plant are discharged on a controlled basis to the downstream polishing pond. It was intended that the sludge would provide a carbon source in the polishing pond to allow denitrification to occur. At the same time, the need for sludge handling, storage, haulage and disposal would be eliminated (Lewandowski and Herskowitz, 1986). Based on early evaluations of the process as operated in Sutton and Port Colborne (Herskowitz, 1987), it was promoted on the basis that it provided the dual benefits of:

i) improved effluent quality in terms of hydrogen sulphide, ammonia, nitrates and bacteria; and,
ii) reduced costs as little or no sludge haulage or external carbon source for denitrification is required.

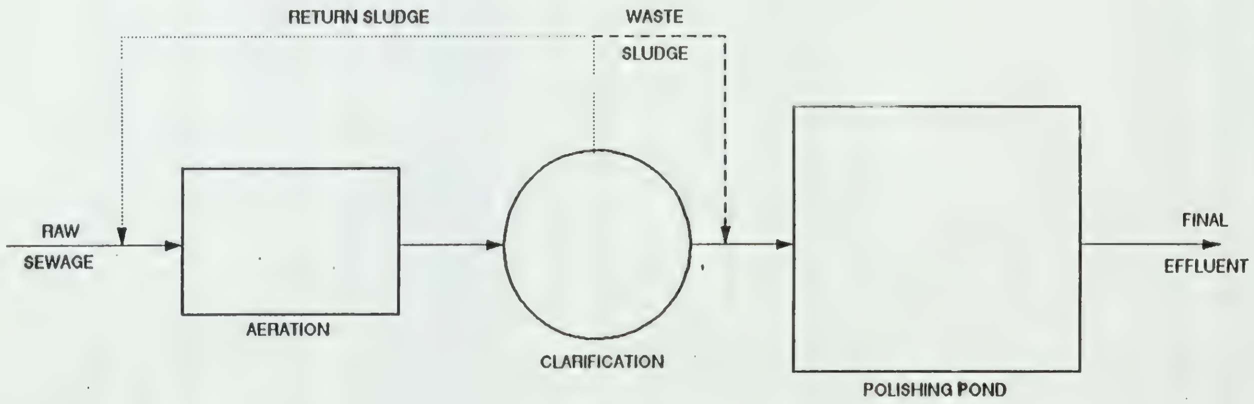


Figure 3.1 Schematic of Sutton Process

3.2.2 Design of Sutton Process Plants

Table 3.3 presents a summary of the design of the eight existing Sutton Process plants. All of the examples of Sutton Process plants included in this review, with the exception of the Cookstown WPCP, represent retrofits to upgrade the effluent quality from an existing facultative lagoon system. At the Cookstown site, new preliminary treatment works, an extended aeration plant and two parallel polishing/storage ponds were constructed in 1988.

In comparing the designs of the various examples of Sutton Process plants, it is important to recognize that different effluent quality requirements have been imposed on these plants. These effluent quality requirements, as summarized in Table 3.4, have an influence on the process designs applied. For example, the Sutton and Lindsay WPCPs do not have effluent ammonia nitrogen limits, whereas all other facilities have ammonia limits specified in their Certificates of Approval. Several of the facilities (Lindsay, Dutton and Rodney) have effluent total phosphorus limits more stringent than 1.0 mg/L at least part of the year. In addition, both Cookstown and Stayner have effluent flow limitations which require some amount of effluent storage in the polishing ponds. This requirement has a direct influence on the volume of the polishing ponds installed at these sites. It should also be noted that none of the Sutton Process plants are required to denitrify to meet nitrate, nitrite or total nitrogen limits.

3.2.2.1 Preliminary Treatment

Of the eight Sutton Process plants included in this review, only the Cookstown WPCP incorporates full preliminary treatment including coarse screening, comminution and grit removal. It is the only one of the eight plants which provides grit removal upstream of the extended aeration plant. Six plants provide coarse screening prior to the extended aeration plant. The Sutton WPCP provides no preliminary treatment upstream of the extended aeration plant.

3.2.2.2 Extended Aeration Design

The design hydraulic retention times (HRTs) in the aeration basins of the extended aeration plants range from a minimum of 16 hours at the Colborne WPCP to a maximum of 55 hours at the Stayner WPCP. The design HRT at Colborne is approximately equal to the minimum HRT of 15 hours suggested by MOE Guidelines (MOE, 1984) for extended aeration facilities. Design HRTs at the other six plants range from 24 hours to 35 hours, averaging about 30 hours.

Few design reports for the Sutton Process plants specify design SRTs or organic loadings. In Table 3.3, the volumetric BOD_5 loading stated in the original design is reported if available. In situations where this design parameter was not available, the volumetric loading was calculated based on the plant design flow and the wastewater strength received at the plant. The volumetric loadings range from a low of 118 kg BOD/1000 m^3.d at the Lindsay WPCP to a high of 256 kg BOD/1000 m^3.d at the Stayner WPCP. The higher loading at the Stayner WPCP reflects the seasonal organic loading imposed on the facility by a local industry which is also reflected by the longer design HRT at this plant compared to the other Sutton Process plants. The overall average design organic loading for the Sutton Process plants is approximately 170 kg BOD_5/1000 m^3.d. This is at the lower range of organic loadings suggested in MOE Guidelines (MOE, 1984) for extended aeration plants (170 to 240 kg/1000 m^3.d)

In six of the eight Sutton plants, aeration is provided by fixed mounted low-speed mechanical surface aerators. The Lindsay WPCP utilizes a combination of aspirating floating mechanical aerators and floating high speed aerators. The high speed floating aerators had been used in the original aerated lagoon at Lindsay and were incorporated into the upgraded design as a cost saving measure. The Cookstown WPCP was designed with five aspirating floating mechanical aerators around the annular aeration basin. Subsequently, two submerged aspirating aerators were added to the aeration basin to provide standby aeration during periods when the floating aerators were out-of-service. The aerator name-plate power per unit volume of aeration basin are generally

TABLE: 3.3
SUMMARY OF DESIGN CRITERIA
SUTTON PROCESS PLANTS

PLANT :	SUTTON	LINDSAY	DUTTON	RODNEY	COLBORNE	TOTTENHAM	STAYNER	COOKSTOWN	MOE DESIGN GUIDELINE (MOE 1984)
DESIGN CAPACITY (m^3/d)	2 046	15 870 (peak) 22 220	558 (peak) 1 885	590 (peak) 2 190	1 375 (peak) 4 626	2 257 (peak) 7 960	1875	825 (peak) 2 634	
YEAR OF START-UP	1981	1990	1991	1991	1983	1988	1991	1986	
DESIGN :									
PRELIM. TREATMENT									
SCREENING	NO	coarse	coarse	coarse	coarse	coarse	coarse	coarse	
GRIT REMOVAL	NO	NO	NO	NO	NO	NO	NO	YES	
COMMINUTOR	NO	NO	NO	NO	YES	NO	NO	YES	
AERATION									
DESIGN HRT (hrs)	24	34 24.0 (peak)	35	33	16	24	55	28	>15
OPERATING SRT		30 to 40			10 to 30 per O&M				>15
summer (d)	70								
winter (d)	150								
DESIGN ORGANIC LOADING kg/1000m3*d	131.7 *	117.9	113.4 **	200.0 **	206.1	170.0	255.7 **	145.5	170 to 240
AERATION METHOD	low speed surface aeration	AIRE-O2 and high speed WELLES	low speed surface aeration	low speed surface aeration	low speed surface aeration	low speed surface aeration	low speed surface aeration	AIRE-O2	
kW	18.6	148.6 and 52.2	45.6	44.7	11.2	37.2	44.7	12.5	
kW/1000 m^3 TANKAGE	9.1	9.0	56.5	55.1	12.3	16.5	10.4	13.0	16 to 25
CLARIFICATION									
NUMBER	ONE	ONE	ONE	ONE	ONE	ONE	TWO	ONE	
SOR @ DESIGN FLOW (m^3/m^2 d)	17.0	17.0	7.5	7.9	8.9	10.1	4.7	10.5	
SOR @ PEAK FLOW (m^3/m^2 d)		24.3	25.2	29.3	31.6	35.0		33.6	35.4
RAS RANGE	UP TO 125%	UP TO 100% peak	UP TO 200%	UP TO 200%	100%	100 to 200%	100%	50 to 200%	50 to 200%
SURFACE SKIMMER	NO	NO	removed	YES	NO	NO	NO	NO	
PHOSPHORUS REMOVAL									
CHEMICAL	ALUM	ALUM	ALUM	ALUM	ALUM	ALUM	ALUM	ALUM	
ADDITION POINTS	aeration effluent batch lagoon	aeration & clarifier effluents	aeration & clarifier effluents	aeration & clarifier effluents	aeration & clarifier effluents	aeration effluent	aeration influent or effluent	aeration effluent	
LAGOONS									
NO. OF CELLS	ONE	SIX	ONE	ONE	ONE	FOUR	FOUR	TWO	
SURFACE AREA (ha/1000 m^3/d)	3.2	3.0	7.3	11.0	2.3	5.0	11.6	8.0	
HRT @ DESIGN FLOW (d)	44	54	66	40	33	75	192	122	
LAGOON RECEIVING SLUDGE***									
SURFACE AREA (ha)	N/A	16.0	N/A	N/A	N/A	5.0	3.9	N/A	
HRT @ DESIGN FLOW (d)		18				55			
SLUDGE HOLDING TANK	YES	YES	NO	NO	NO	NO	NO	NO	

* Based on 1990 [BODinf]

** Based on 1989 [BODinf]

*** In plants with more than one lagoon, where not all lagoons receive waste sludge from the extended aeration plant, this design information refers only to those lagoons which do receive sludge.

TABLE 3.4
SUMMARY OF SUTTON PROCESS PLANTS
EFFLUENT QUALITY PARAMETERS*

PLANT :	SUTTON	LINDSAY	DUTTON	RODNEY	COLBORNE	TOTTENHAM	STAYNER	COOKSTOWN
DESIGN CAPACITY (m3/d)	2 046	15 870 (peak) 22 220	558 (peak) 1 885	590 (peak) 2 190	1 375 (peak) 4 826	2 257 (peak) 7 960	1 875	825 (peak) 2 634
YEAR OF START–UP	1981	1990	1991	1991	1983	1988	1991	1988
Effluent Limits:								
BOD (mg/L)	annual 25.00	annual 15.00	winter /summer monthly 15.00 10.00 daily 25.00 15.00	winter /summer monthly 15.00 10.00 daily 25.00 15.00	annual 15.00	annual 7.00	annual 15.00	annual 25.00
LOADING (kg/d)			6.51	3.90				
TSS (mg/L)	annual 25.00	annual 15.00	monthly 15.00 10.00 daily 25.00 15.00	monthly 15.00 10.00 daily 25.00 15.00	annual 25.00	annual 15.00	annual 15.00	annual 25.00
LOADING (kg/d)			6.51	3.90				
TP (mg/L)	monthly 1.00	monthly 0.30	monthly 1.00 0.50 daily 1.50 1.00	monthly 1.00 0.50 daily 1.50 1.00	monthly 1.00	monthly 1.00	arithmetic mean of 0.50 any two consecutive samples	monthly 1.00
LOADING (kg/d)		1400 kg/yr	0.37	0.28		820 kg/yr	annual 320 kg/yr	300 kg/yr
T NH4 (mg/L)			monthly 5.00 3.00 daily 7.50 4.50	monthly 5.00 3.00 daily 7.50 4.50	May 15–Oct 1 2.00 other times: @ 15 Cels. 3.00 @ 10 Cels. 4.50 @ 5 Cels. 6.80	* Dec–Mar 4.70 **Apr–May & **Oct–Nov 1.60 * Jun–Sept 0.70 * 4–month arith. mean ** 2–month arith. mean	Apr. 1–May 15 2.00 Sep 16–Oct 31 2.00 Nov 1–Mar 31 4.00	annual 4.00 OR monthly for: Apr/Oct/Nov
LOADING (kg/d)			2.05	1.57				
OTHER		C. of A. capacity is 12 672 m^3/d		Total residual chlorine: Monthly 0.01 mg/L Daily 0.03 mg/L	Max. capacity is 4 826 m^3/d		TP revised to 1.0 mg/L, (1990) Dilution ratio of effluent to Creek flow does not exceed 1:3	Discharge: April 60 L/s Oct–Mar 10 L/s May 10 L/s No Discharge from June 1–Sept 30

* For additional detail refer to appendix 1

in the range from about 9 to 16 kW/1000 m^3. This is at the low end of the range suggested by MOE Guidelines (MOE, 1984) for mechanical aeration systems (16 to 25 kW/1000 m^3). The exceptions are the similarly designed Rodney and Dutton WPCPs which provide in excess of 50 kW of nameplate aerator power per 1000 m^3 of aeration basin volume.

Aeration basin geometry and construction techniques vary considerably from plant to plant. The Sutton WPCP and Tottenham WPCP aeration basins are lined earthen basins while the Lindsay WPCP aeration was constructed from the original aerated lagoon which had previously been at the site. The Cookstown WPCP is constructed with an annular aeration basin around a central circular clarifier (refer to Appendix 1, Figure A1.22). The other four plants have traditional concrete aeration tanks.

All of the Sutton Process plants with the exception of Stayner have a single clarifier associated with the extended aeration plant. The clarifiers were designed with peak surface overflow rates in the range from about 25 to 35 m^3/m^2.d. These are conservative overflow rates compared to MOE Guidelines (MOE, 1984) which recommend peak overflow rates of 35 m^3/m^2.d for extended aeration facilities operated with or without phosphorus removal. The design peaking factor for the Sutton WPCP and the Stayner WPCP were not available. Based on the average day clarifier loading at the Sutton WPCP (17.0 m^3/m^2.d), this clarifier sizing is consistent with the other six plant. The Stayner WPCP was designed with an average day clarifier surface loading of about 5 m^3/m^2.d which would provide a very conservative peak day loading of 15 m^3/m^2.d based on a peaking factor of 3.0.

Because the Sutton WPCP was originally designed to operate as a conventional extended aeration facility with sludge disposal to agricultural land, an aerated holding tank was included in the original design. Until 1991, this aerated holding tank was not used as sludge was discharged to the polishing lagoon according to the Sutton Process operating mode. In 1991, the plant began to waste excess biomass from the extended aeration plant to the aerated holding tank in an attempt to further stabilize the sludge prior to its

discharge into the polishing pond. This operating procedure was adopted in an attempt to reduce the apparent solubilization of phosphorus and ammonia nitrogen in the pond (refer to Section 3.2.5). Of the other seven Sutton Process plants, only the Lindsay WPCP has a sludge holding tank. This unmixed tank was constructed from part of one of the original aerated lagoons at the facility. At start-up, the sludge holding tank was used to store excess sludge prior to its release into the polishing pond. This practice has been discontinued because of problems with odours produced in the sludge holding tank.

3.2.2.3 Phosphorus Removal

All eight Sutton Process plants practise phosphorus removal. Phosphorus precipitation is achieved by alum addition to the extended aeration plant. The dosage point is either the aeration basin or the channel feeding the secondary clarifier. All of the plants with the exception of Cookstown and Tottenham also are equipped to add alum to the secondary clarifier effluent flow to the polishing lagoon. Experimentation at Sutton and at Colborne by MOE did not suggest that performance improvements were achieved by chemical addition to the clarifier effluent on a continuous basis. Therefore, none of the plants routinely add chemicals to the influent to the polishing pond. At the Sutton WPCP, the lagoon has been "batch" treated with alum on several occasions since 1987 in response to elevated concentrations of phosphorus in the lagoon. Batch treatment is done by adding a large batch dosage of alum to the lagoon influent over a short time period.

In addition to alum addition for chemical precipitation of phosphorus, the Rodney and Dutton WPCPs are designed to allow lime addition to the aeration basin to supplement the alkalinity content of the wastewater if needed.

3.2.2.4 Lagoon Design

As noted earlier, all of the Sutton Process plants with the exception of the Cookstown WPCP were implemented as upgrades to an existing waste stabilization lagoon. In general, minimal modifications to the existing lagoons were made at the time that the

Sutton Process was implemented. Therefore, the lagoon designs at these plants reflect the original lagoon design more than any conscious design philosophy for the Sutton polishing pond.

At the Sutton, Lindsay, Dutton, Rodney and Colborne WPCP, only minor changes to the location of influent or effluent piping at the lagoon or changes to the operating liquid level were made when the Sutton Process was implemented. No sludge was removed from these lagoons prior to modifying the system for Sutton Process operation. At both Tottenham and Stayner, the polishing ponds receiving sludge were dewatered and deepened in the area where the sludge from the extended aeration plant was discharged. These deeper "pods" were intended to keep the accumulated sludge in a localized area rather than distributing it over the entire pond. No sludge was removed from the lagoons at Tottenham when the plants were converted to the Sutton Process.

Four of the eight Sutton Process plants have a single polishing pond which receives both extended aeration plant effluent and wasted biological sludge. These are the Sutton WPCP, the Dutton WPCP, the Rodney WPCP and the Colborne WPCP. The Cookstown WPCP has two parallel ponds which were designed to each receive half of the wasted sludge from the extended aeration plant. The other three systems have multiple polishing ponds and not all of the ponds receive sludge. At the Lindsay WPCP, there are two parallel trains of three lagoons and only the first lagoon in each train receives sludge. Similarly at the Tottenham WPCP, the first lagoons in each parallel set of two lagoons receive sludge. At the Stayner WPCP, either of the first two of four lagoons can receive clarifier effluent but only one lagoon can receive sludge. At both Lindsay and Cookstown, plant staff feel that one of the two lagoons receives more sludge than the other because of differences in liquid level or piping length.

At most of the Sutton Process plants, a common pipe transports clarifier effluent and waste sludge to the lagoons. This is the case at the Sutton WPCP, the Colborne WPCP, the Lindsay WPCP, the Dutton WPCP, and the Tottenham WPCP. At the Cookstown WPCP, the sludge enters the polishing lagoons through separate pipes which discharge

near the same points as the clarifier effluent discharge. At the Rodney WPCP and the Stayner WPCP, sludge enters the lagoon through a separate pipe from the clarifier effluent and at a different location.

The design HRT in the lagoons range from a low of 33 days at the Colborne WPCP to a high of 122 days at the Cookstown WPCP and 192 days at the Stayner WPCP. At the latter two plants, there are seasonal restrictions on the plant volumetric discharges which necessitate treated effluent storage. Hence, the ponds serve to polish the extended aeration plant effluents and to provide storage. The longer HRT provided in these facilities reflects the storage requirement rather than the polishing requirement. For Sutton Process plants which are not required to provide seasonal storage, the HRT in the polishing ponds range from 33 days to 75 days, averaging about 55 days. At the Lindsay WPCP where only two of the six ponds receive sludge, the ponds receiving sludge only provide about 18 days retention at design flow. Several of the lagoons, including those at Sutton, Rodney, Dutton and Colborne, have influent and effluent piping locations which would allow some short-circuiting, reducing the actual HRT in these polishing ponds.

3.2.3 Operation and Performance of Sutton Process Plants

The recent operating and performance history (1989, 1990/91) for the five Sutton Process plants which were in operation during this period (Sutton WPCP, Colborne WPCP, Tottenham WPCP, Cookstown WPCP and Lindsay WPCP) is summarized in Table 3.5.

3.2.3.1 Operating Conditions

Only three Sutton Process plants - Sutton WPCP, Colborne WPCP and Tottenham WPCP- have an extensive history of operation having been commissioned as Sutton plants in 1981, 1983 and 1988, respectively. Therefore, the following analysis is based primarily on the performance of these three plants. The Cookstown WPCP was commissioned at about the same time as the Tottenham WPCP. However, it is currently operating at less than 40 percent of its design capacity and all of the plant flow has been discharged

TABLE: 3.5 CURRENT OPERATING CONDITIONS SUTTON PROCESS PLANTS				
PLANT :	COLBORNE		SUTTON	
YEAR MONTHS	1989	1990	1989	1990 (Jan-Jul)
AVERAGE DAY FLOW (m3)	* 789	* 912	1451	1489
MAXIMUM DAY FLOW (m3) EST.	** 1728	** 1998	** 2786	** 2859
RAW SEWAGE:				
AVG. BOD INF. (mg/L)	87.4	99.0	149.0	133.4
AVG. TSS INF. (mg/L)	154.9	116.0	137.1	149.0
AVG. TKN INF. (mg/L)	N/D	30.0	42.4	38.2
AVG. TP INF. (mg/L)	4.5	4.9	7.0	5.9
AERATION				
OPERATING HRT (hrs)	27.8	24.0	33.8	33.0
OPERATING SRT (d)	73	56	54	58
F/M	0.03	0.03	0.04	0.03
ORGANIC LOADING (kg/1000m3(AERAT.)*d)	75	99	106	97
CLARIFICATION				
OPERATING HRT (hrs)	17.2	14.8	5.3	5.1
SOR @ AVERAGE FLOW (m ^3/m ^2 d)	5.2	6.0	12.1	12.4
SOR @ PEAK DAY FLOW (m ^3/m ^2 d)	** 11.33	** 13.1	** 23.2	** 23.8
SECONDARY EFFLUENT:				
AVG. BOD (mg/L)	5.4	4.9	14.9	14.1
STD.DEV.	3.0	1.2	10.4	9.5
AVG. TSS (mg/L)	13.1	9.2	9.2	9.7
STD.DEV.	7.9	2.8	3.9	1.7
AVG. TKN (mg/L)	1.0	0.8	5.3	2.8
STD.DEV.	0.4	0.2	6.3	2.1
AVG. NH3-N (mg/L)	0.1	0.1	4.0	1.7
STD.DEV.	0.03	0.03	6.1	1.8
AVG. NO(T)-N (mg/L)	14.7	12.8	N/D	N/D
STD.DEV.	2.6	1.6	N/D	N/D
AVG. TP (mg/L)	0.7	0.5	0.2	0.3
STD.DEV.	0.2	0.2	0.1	0.2
LAGOON (receiving sludge)				
OPERATING HRT (d)	67	58	81	79
LAGOON EFFLUENT:				
AVG. BOD (mg/L)	3.2	3.7	6.2	5.8
STD.DEV.	2.4	5.3	6.4	2.5
AVG. TSS (mg/L)	4.3	6.8	9.3	8.9
STD.DEV.	2.3	12.3	10.5	5.6
AVG. TKN (mg/L)	3.3	1.6	7.8	10.7
STD.DEV.	2.6	1.0	5.8	6.4
AVG. NH3-N (mg/L)	2.0	4.0	6.9	8.7
STD.DEV.	2.4	0.7	4.6	5.8
AVG. NO(T)-N (mg/L)	4.0	2.9	N/D	N/D
STD.DEV.	3.4	2.7	N/D	N/D
AVG. TP (mg/L)	0.5	0.3	0.6	0.4
STD.DEV.	0.3	0.2	0.7	0.3
SLUDGE				
ESTIMATED ANNUAL SLUDGE LOADING ON LAGOON (kg/m3 of lagoon) receiving sludge	0.4	0.5	0.5	0.5
ESTIMATED SLUDGE PRODUCED (kg/d)	56.6	75.1	170.2	155.4
TOTAL ESTIMATED SLUDGE DISCHARGED TO LAGOON (tonnes) (since start-up)	216		642	

* correction factor for flow recorder error included

** P.F. based on 1991 data to estimate peak day flow

TABLE 3.5 (cont)
CURRENT OPERATING CONDITIONS
SUTTON PROCESS PLANTS

PLANT	TOTTENHAM		COOKSTOWN		LINDSAY
YEAR	1990	1991	1989	1990	1991
MONTHS		(Jan–Sep)			
AVERAGE DAY FLOW (m3)	1545	1475	248	389	12577
MAXIMUM DAY FLOW (m3)	4510	4663	613	719	37160
RAW SEWAGE:					
AVG. BOD INF. (mg/L)	203.0	128.5	148.5	194.4	79.9
AVG. TSS INF. (mg/L)	324.0	191.1	227.1	229.2	99.9
AVG. TKN INF. (mg/L)	34.2	27.8	N/D	52.9	24.4
AVG. TP INF. (mg/L)	10.0	5.6	7.8	8.3	3.6
AERATION					
OPERATING HRT (hrs)	35.06	36.7	93.0	59.4	42.4
OPERATING SRT (d)	45.17	75	148	72	93
F/M	0.04	0.03	0.01	0.03	0.02
ORGANIC LOADING (kg/1000m3(AERAT.)*d)	139	84	38	79	45
CLARIFICATION					
OPERATING HRT (hrs)	12.5	13.1	25.0	16.0	7.1
SOR @ AVERAGE FLOW (m^3/m^2 d)	6.9	6.6	3.5	5.5	13.5
SOR @ PEAK DAY FLOW (m^3/m^2 d)	20.2	20.9	8.5	10.0	39.7
SECONDARY EFFLUENT:					
AVG. BOD (mg/L)	7.3	4.6	4.6	7.9	* 5.0
STD.DEV.	4.2	5.4	1.6	3.5	3.6
AVG. TSS (mg/L)	14.1	6.0	14.9	15.5	7.3
STD.DEV.	9.9	4.4	6.5	9.1	4.1
AVG. TKN (mg/L)	1.1	0.8	1.2	1.2	1.1
STD.DEV.	0.3	0.2	0.5	0.4	0.4
AVG. NH3–N (mg/L)	N/D	0.1	0.2	0.1	0.2
STD.DEV.	N/D	0.1	0.4	0.0	0.3
AVG. NO(T)–N (mg/L)	21.3	21.1	23.0	18.7	10.3
STD.DEV.	5.9	2.8	7.8	2.3	4.5
AVG. TP (mg/L)	1.4	0.6	0.9	0.5	0.6
STD.DEV.	0.7	0.3	0.2	0.4	0.3
LAGOON (receiving sludge)					
OPERATING HRT (d)	49	52	405.4	258.9	23
LAGOON EFFLUENT:					
AVG. BOD (mg/L)	3.2	2.5	2.3	8.0	3.3
STD.DEV.	3.4	2.0	1.9	4.6	2.5
AVG. TSS (mg/L)	6.5	3.3	5.0	N/D	7.2
STD.DEV.	7.9	2.6	4.0	N/D	6.0
AVG. TKN (mg/L)	1.5	1.4	N/D	2.7	1.7
STD.DEV.	0.5	0.4	N/D	1.2	0.6
AVG. NH3–N (mg/L)	0.3	0.2	N/D	0.9	0.5
STD.DEV.	0.3	0.2	N/D	1.2	0.5
AVG. NO(T)–N (mg/L)	3.3	2.8	N/D	N/D	6.3
STD.DEV.	3.7	3.2	N/D	N/D	3.4
AVG. TP (mg/L)	0.5	0.3	0.3	0.3	0.2
STD.DEV.	0.2	0.1	0.2	0.1	0.1
SLUDGE					
ESTIMATED ANNUAL SLUDGE LOADING ON LAGOON (kg/m3 of lagoon) receiving sludge	1.3	0.8	0.1	0.2	1.1
ESTIMATED SLUDGE PRODUCED (kg/d)	274.8	165.9	32.3	66.1	824.0
TOTAL ESTIMATED SLUDGE DISCHARGED TO LAGOON (tonnes) (since start–up)	322		72		8

* Linsday lagoon effluent average of north and south outfa

during a very short period in the spring when the maximum discharge rate is allowed. Because of this practice, there is very limited final effluent quality data on which to base a performance evaluation. The Lindsay WPCP was commissioned in 1990; therefore, the data presented in Table 3.5 represents the first full year of plant operation.

The Colborne WPCP, the Sutton WPCP and the Tottenham WPCP were operating at between about 60 and 75 percent of their design capacity during the period summarized in Table 3.5. (The flows to the Colborne WPCP have been corrected to account for flow metering errors - refer to Appendix 1, Section A1.2). Therefore, the aeration basin hydraulic loadings and clarifier surface overflow rates have been substantially lower than design. HRTs in aeration basins have ranged from about 24 hours to 36 hours and estimated peak day clarifier loadings have ranged from about 11 $m^3/m^2.d$ to about 24 $m^3/m^2.d$.

Based on average MLSS concentrations, the extended aeration systems at these plants have operated at organic loadings of less than 0.05 g BOD_5/g VSS.d. These plants do not measure and record the volumes and concentrations of waste sludge diverted to the polishing ponds. Therefore, there are no data on which to base a calculated SRT. The SRT reported in Table 3.5 is based on the organic loading and an estimated yield factor of 0.65 g VSS/g BOD removed. Based on this yield, the SRT at these plants is estimated to range between about 50 and 75 days.

3.2.3.2 Extended Aeration Plant Effluent Quality

Under the operating conditions described above at the Sutton WPCP, Colborne WPCP and Tottenham WPCP, the extended aeration plants at these three facilities have produced a well-nitrified effluent containing generally less than 1.0 mg/L NH_3-N on an annual average basis. The exception was the Sutton WPCP where less consistent nitrification was reported. In 1989, the extended aeration plant effluent contained an average NH_3-N concentration of 4.0 mg/L (standard deviation 6.1 mg/L). An improvement in nitrification was apparent in 1990 when the average effluent quality was

1.7 mg/L NH_3-N (standard deviation 1.8 mg/L). The Sutton WPCP has historically received large quantities of hauled sewage which is discharged directly to the plant aeration basin. In 1990, a total volume of 5000 m^3 of hauled sewage was disposed to the Sutton WPCP. The exact impact of this loading on the operation of the extended aeration plant cannot be defined based on available data; however, it is possible that the short term increase in loading on the aeration basin may result in impairment of nitrification efficiency. Based on limited data from the Cookstown and Lindsay WPCPs, these two plants also achieved virtually complete nitrification producing effluents containing less than 1.0 mg/L NH_3-N on average.

Average effluent TSS concentrations at these three extended aeration plants were less than 15 mg/L on an annual basis, reflective of the low surface loading rates. The Tottenham WPCP extended aeration effluent did not achieve an annual average TP concentration of 1.0 mg/L in 1990. With this exception, the extended aeration plant effluents all contained less than 1.0 mg/L TP on average.

3.2.3.3 Final Effluent Quality

Average lagoon effluent quality is included in Table 3.5 for the five facilities with an operating history. More detailed data are included in Appendix 1.

The polishing effect of the lagoon on the extended aeration plant effluent is apparent when the final effluent BOD_5 and TSS are compared to the extended aeration plant effluent concentrations for the same parameters. The Sutton Process plants for which data are presented in Table 3.5 produced tertiary quality effluents containing average BOD_5 and TSS concentrations less than 10 mg/L.

A similar improvement in the TP content of the extended aeration plant effluent after polishing in the lagoons is apparent at all of the Sutton Process plants except the Sutton WPCP. Total phosphorus concentrations in the other lagoon effluents generally averaged less than 0.5 mg/L. The lagoon effluent at the Sutton WPCP actually contained higher

phosphorus concentrations on average than the extended aeration plant effluent in both 1989 and 1990. This increase in the phosphorus concentration of the lagoon was noticeable at the Sutton WPCP as early as 1987 when plant staff initiated batch addition of alum to the lagoon to reduce the phosphorus concentration prior to discharge. Figure 3.2 compares the TP content of the lagoon effluent with that of the extended aeration plant since the Sutton WPCP began operating in the Sutton Process mode in 1981. After an initial period of erratic phosphorus removal performance (1981/82), occasional high TP concentrations are evident in the lagoon effluent in 1983 and in subsequent years. In August 1987, the lagoon was batch-treated with alum to reduce the phosphorus concentration from levels in excess of 1.0 mg/L. No chemical treatment of the lagoon was done in 1988 or 1989 despite occasional TP concentrations in excess of 2.0 mg/L. In 1990, chemical addition to the lagoon was done in April, May, July, August and October. The lagoon was batch-treated again in September of 1991. There are insufficient data available to isolate the cause of increased phosphorus concentrations in the polishing pond at the Sutton WPCP. Limited sampling within the sludge pile did not identify elevated soluble phosphorus concentrations (refer to Section 3.2.5.3) although the batch addition of alum in September 1991 directly to the clarifier outfall may have affected the results. None of the other Sutton plants have exhibited increasing TP concentrations in the polishing pond. It is possible that the release is specifically related to the age of the sludge at Sutton, the water chemistry or the relative amounts of biologically and chemically bound phosphorus present in the sludge.

The occurrence of denitrification is apparent at the plants where nitrate and nitrite data are available (Colborne, Tottenham and Lindsay). On average, about 3 to 5 mg/L of oxidized nitrogen (nitrate plus nitrite) is present in the lagoon effluent compared to 15 to 20 mg/L in the extended aeration plant effluents.

There is a consistent deterioration in final effluent quality compared to extended aeration effluent in terms of both ammonia and TKN. At the newer Sutton Process plants (Tottenham, Cookstown and Lindsay), the increase in ammonia and TKN concentrations across the polishing pond are typically less than 1 mg/L. At both of the older facilities

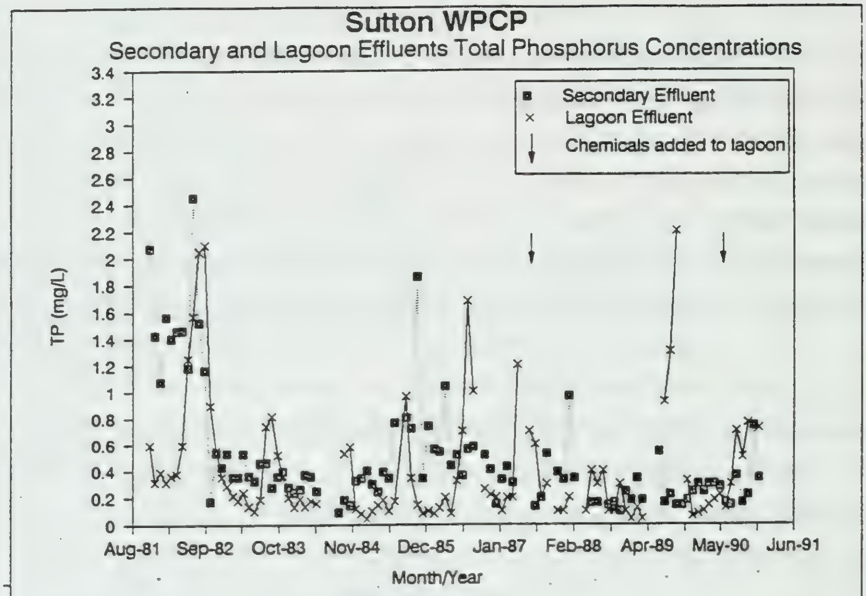

Sutton WPCP
Secondary and Lagoon Effluents Total Phosphorus Concentrations
Secondary Effluent
Lagoon Effluent
Chemicals added to lagoon
TP (mg/L)
3.4
3.2
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
Aug-81
Sep-82
Oct-83
Nov-84
Dec-85
Jan-87
Feb-88
Apr-89
May-90
Jun-91
Month/Year

Figure 3.2

(Sutton WPCP and Colborne WPCP), the increase in both ammonia and TKN is more significant. Figures 3.3 and 3.4 present the chronology of extended aeration and lagoon effluent NH_3-N and TKN concentrations at the Sutton WPCP since 1981. Up until about 1986, there was no clear pattern of elevated ammonia or TKN concentrations in the lagoon effluent compared to the extended aeration plant effluent. By 1987, higher nitrogen concentrations became evident and by 1990, the ammonia content of the pond effluent exceeded 15 mg/L NH_3-N despite extended aeration effluent concentrations consistently below 5 mg/L. Similarly, the final effluent from the Colborne WPCP began to show elevated ammonia concentrations in 1990 and 1991 compared to the consistent low concentrations in the extended aeration plant effluent (Figure 3.5). TKN data show a similar pattern (Figure 3.6). The high nitrogen concentrations in Figures 3.5 and 3.6 in 1988 at the Colborne WPCP reflect a period when raw sewage was diverted directly to the lagoon so that clarifier maintenance could be undertaken. Earlier high nitrogen concentrations in 1983 probably reflect start-up of the Sutton Process at Colborne and flush-out of the original lagoon contents.

The Sutton and Colborne plant data suggest that significant increases in the ammonia content of the polishing pond begin to occur after about six to seven years of operation. However, smaller increases in ammonia concentrations are apparent immediately in these polishing ponds. Based on samples collected within the sludge piles at the Sutton WPCP, the Colborne WPCP and the Tottenham WPCP (refer to Section 3.2.5.3), ammonia is being solubilized from the biomass under anaerobic conditions. The rapidity and magnitude of the ammonia concentration increase in the polishing ponds will be affected by the amount of sludge deposited in the lagoon relative to its overall liquid volume.

Limited data are available from the Sutton Process plants to define their ability to control bacterial levels without chemical disinfection. Data collected by the MOE during their intensive evaluations of the Sutton Process as operated at the Sutton WPCP (Lewandowski and Herskowitz, 1986) and the Colborne WPCP (Herskowitz, 1987) suggest that fecal bacteria reductions of between 95 to more than 99 percent were achieved in the pond after initial reductions of a similar magnitude in the extended

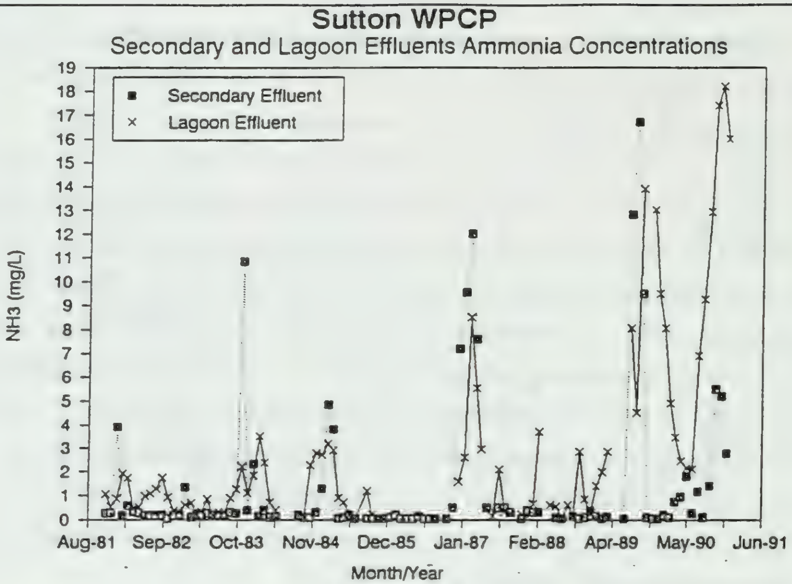


Figure 3.3

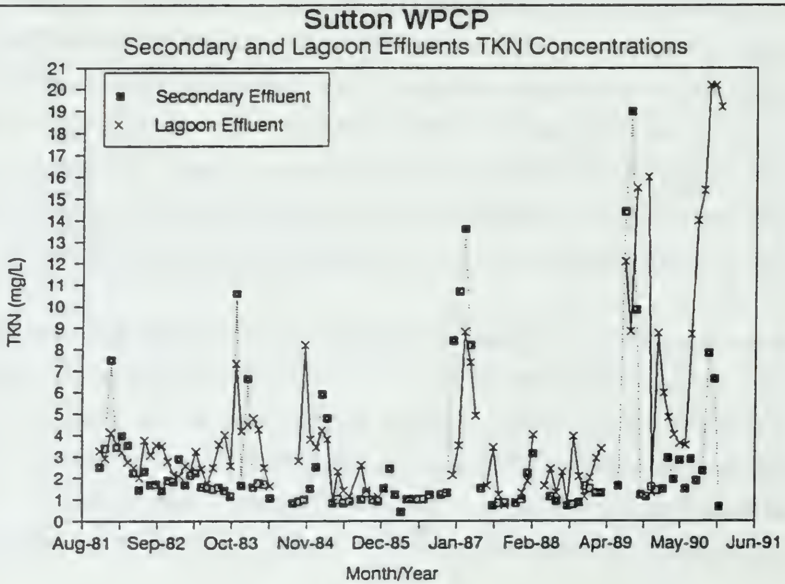


Figure 3.4

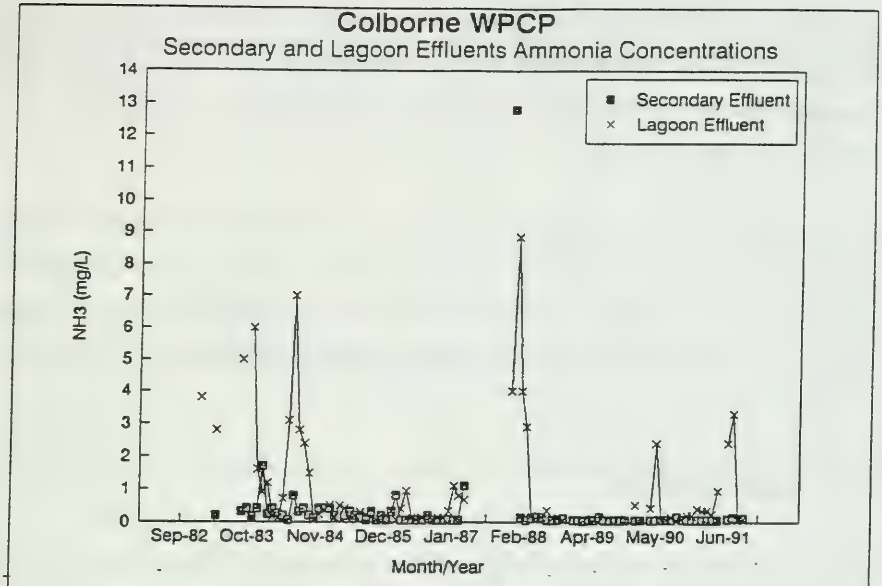


Figure 3.5

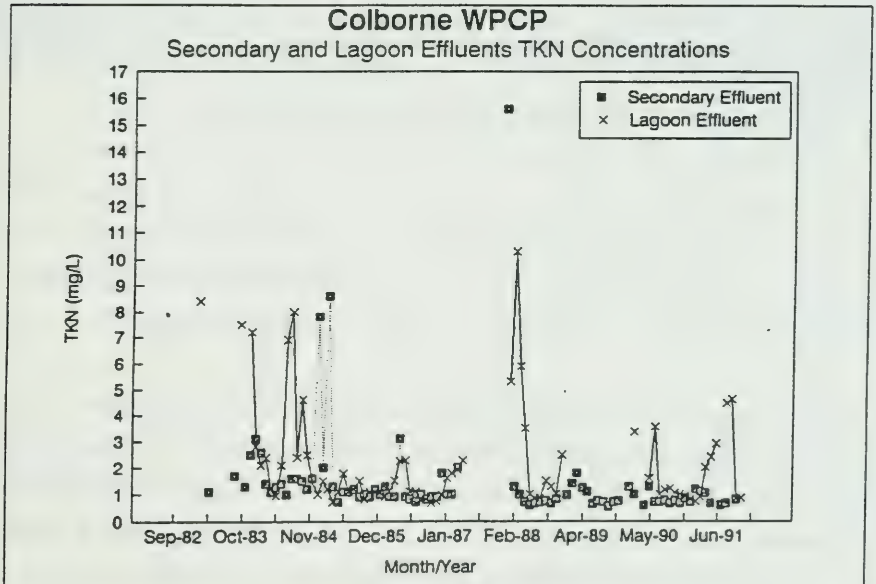


Figure 3.6

aeration plant. At the Colborne WPCP, the geometric mean fecal coliform and Pseudomonas aeruginosa contents of the pond effluent were 51 per 100 ml and 3 per 100 ml respectively. Elevated bacterial levels were noted at both Sutton and Colborne in late winter (February to March).

Hydrogen sulphide was never detected at concentrations above the detection limit in either Sutton or Colborne pond effluents during the intensive MOE monitoring done at these plants. None of the plants reported odour problems during the spring break-up which had been common at most of these lagoons prior to implementation of the Sutton Process.

3.2.3.4 Seasonal Effects

Only the Sutton WPCP, the Colborne WPCP and the Tottenham WPCP provided sufficient data to allow a seasonal comparison of secondary and final effluent quality. For these three plants, this comparison is provided in Tables 3.6, 3.7 and 3.8.

In terms of total phosphorus and BOD_5, there is no apparent seasonal trend in either the secondary plant effluents or the polishing pond effluents. All three extended aeration plants produced higher TSS concentrations in the winter months than in the other three seasons; however, there is no indication of a seasonal effect on TSS in the lagoon effluent. In general, TSS concentrations in the lagoon effluents are lower in the summer and fall when conventional lagoons typical display high TSS concentrations due to algal blooms. Sutton Process polishing ponds appear to be less susceptible to algal blooms than conventional lagoons, as was noted by many of the plant operating staffs.

In terms of nitrogen concentrations (NH_3-N and TKN), the Sutton and Tottenham WPCPs did not show any significant reduction in nitrification efficiency in the winter months. The data from the Colborne WPCP suggest that nitrification is less effective in the winter than in other periods, based on extended aeration plant effluent concentrations averaging 1.8 mg/L. However, the winter average is skewed by data from the winter of 1988 when

TABLE: 3.6
SUTTON WPCP
SEASONAL DATA FOR 1982–1990

	TP SECONDARY EFFLUENT mg/L	TP LAGOON EFFLUENT mg/L	NH3–N SECONDARY EFFLUENT mg/L	NH3–N LAGOON EFFLUENT mg/L	BOD SECONDARY EFFLUENT mg/L	BOD LAGOON EFFLUENT mg/L	TSS SECONDARY EFFLUENT mg/L	TSS LAGOON EFFLUENT mg/L	TKN SECONDARY EFFLUENT mg/L	TKN LAGOON EFFLUENT mg/L
WINTER										
AVG	0.48	0.16	1.28	1.99	8.94	3.48	13.99	4.68	2.90	2.96
STD DEV	0.33	0.09	1.71	2.13	1.96	1.52	6.64	2.38	1.76	1.36
SPRING										
AVG	0.48	0.21	1.51	2.18	10.37	5.06	13.03	6.77	3.07	3.48
STD DEV	0.36	0.15	2.96	1.54	4.75	1.89	8.22	3.15	2.87	1.44
SUMMER										
AVG	0.70	0.69	1.44	2.20	7.06	6.10	6.36	9.60	2.62	4.35
STD DEV	0.61	0.48	3.18	2.54	5.24	4.63	1.64	8.03	3.44	3.39
FALL										
AVG	0.59	0.64	1.22	4.04	7.68	5.12	9.81	7.90	2.29	6.21
STD DEV	0.57	0.33	1.63	5.80	6.52	2.31	8.24	4.60	1.67	6.04

TABLE: 3.7
COLBORNE WPCP
SEASONAL DATA FOR 1983–1991

	TP SECONDARY EFFLUENT mg/L	TP LAGOON EFFLUENT mg/L	NH3–N SECONDARY EFFLUENT mg/L	NH3–N LAGOON EFFLUENT mg/L	BOD SECONDARY EFFLUENT mg/L	BOD LAGOON EFFLUENT mg/L	TSS SECONDARY EFFLUENT mg/L	TSS LAGOON EFFLUENT mg/L	TKN SECONDARY EFFLUENT mg/L	TKN LAGOON EFFLUENT mg/L
WINTER										
AVG	0.87	0.46	1.80	1.67	9.07	5.99	17.19	10.54	3.47	3.31
STD DEV	0.34	0.53	4.17	2.11	4.74	6.26	5.21	15.04	4.68	2.87
SPRING										
AVG	0.67	0.31	0.20	1.11	6.07	4.96	11.96	8.54	1.48	2.17
STD DEV	0.20	0.08	0.16	0.92	1.18	1.42	4.75	6.23	0.76	0.70
SUMMER										
AVG	0.80	0.60	0.14	1.29	4.00	2.97	6.23	4.75	0.85	2.46
STD DEV	0.75	0.44	0.11	1.67	1.38	2.60	1.86	1.57	0.25	1.94
FALL										
AVG	0.77	0.53	0.13	1.05	3.80	2.35	7.90	5.01	0.96	2.19
STD DEV	0.67	0.39	0.07	1.87	1.26	1.35	1.79	3.89	0.31	2.21

TABLE 3.8 TOTTENHAM WPCP SEASONAL DATA FOR 1986–1990										
	TP SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L	NH3–N SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L	BOD SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L	TSS SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L	TKN SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L
WINTER										
AVG	1.59	0.39	ND	0.49	9.21	3.20	20.14	4.74	1.34	1.47
STD DEV	0.67	0.13	ND	0.05	3.23	0.02	4.63	0.46	0.12	0.01
SPRING										
AVG	1.95	0.28	0.15	0.99	10.37	8.87	18.45	14.84	1.54	2.18
STD DEV	0.12	0.10	0.00	0.21	1.80	2.91	3.22	4.26	0.39	0.25
SUMMER										
AVG	0.78	0.62	ND	0.31	4.65	4.68	14.64	7.19	0.86	1.66
STD DEV	0.42	0.23	ND	0.07	1.14	2.69	7.69	3.52	0.27	0.09
FALL										
AVG	1.19	0.24	0.03	0.18	5.67	4.21	10.48	6.90	1.06	1.18
STD DEV	0.34	0.10	0.00	0.00	0.49	4.11	1.25	4.20	0.15	0.09

TABLE 3.9 SUTTON WPCP SEASONAL DATA FOR 1988–1990										
	TP SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L	NH3–N SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L	BOD SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L	TSS SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L	TKN SECONDARY EFFLUENT mg/L	LAGOON EFFLUENT mg/L
WINTER										
AVG	0.26	0.09	0.15	4.35	8.62	2.85	10.91	2.79	1.47	3.67
STD DEV	0.00	0.00	0.03	3.13	1.72	0.18	0.34	0.42	0.00	1.67
SPRING										
AVG	0.24	0.11	0.65	2.77	11.17	3.65	11.25	6.03	1.92	3.74
STD DEV	0.06	0.07	0.52	0.09	1.03	1.55	1.75	2.78	0.61	0.39
SUMMER										
AVG	0.27	0.74	5.18	6.18	14.64	12.64	8.49	21.65	6.86	9.61
STD DEV	0.05	0.37	4.68	0.10	4.96	5.07	1.52	6.85	4.81	0.88
FALL										
AVG	0.26	0.97	3.63	14.81	20.07	4.75	7.61	7.24	4.81	17.18
STD DEV	0.11	0.29	0.40	1.36	1.76	1.15	1.36	1.33	0.76	1.42

the secondary plant was shut down to repair the clarifier and nitrification had to be re-established in the cold weather. With the exception of this period, the secondary plant at Colborne has consistently nitrified efficiently year-round despite the shortest aeration basin HRT of any of the Sutton Process plants.

Similarly, the lagoon effluents at Colborne and Tottenham do not suggest any seasonal trends in terms of effluent NH_3-N or TKN concentrations which would indicate a seasonal effect on nitrogen release from the accumulated sludge. The data from the Sutton WPCP (Table 3.6) does suggest that the highest concentrations of nitrogen are present in the pond in the fall, although the seasonal differences are not significant. The seasonal differences at the Sutton have been more significant since 1988 as shown in Table 3.9. Increases in lagoon effluent TP concentrations in the summer and fall are apparent as well as significant increases in the nitrogen concentrations in the fall. In addition, the data suggest that the lagoon effluent TSS concentrations have begun to rise in the summer and fall periods.

3.2.3.5 Effluent Toxicity

Final effluent samples were collected at the Lindsay, Tottenham, Colborne and Sutton WPCPs in the summer of 1990 and the winter of 1991 by MOE for acute toxicity testing using rainbow trout and Daphnia magna as the test species. The results of these tests are presented in Table 3.10.

All samples analyzed were either non-lethal to rainbow trout or had LC_{50} values in excess of 100 percent. Only summer samples underwent testing for acute toxicity to Daphnia magna. All samples tested using this procedure were either non-lethal or had LC_{50} values greater than 100 percent.

Limited chemical analysis was done on the samples used for toxicity testing. The only chemicals analyzed that could be related to toxicity were ammonia and nitrite. Concentrations of these compounds in all effluents were below levels expected to be

Table 3.10 Sutton Process				
		Toxicity Test Results		NH_3-N Concentration (mg/L)
WPCP	Season	Trout LC_{50}	Daphnia Magna LC_{50}	
Lindsay	Summer	NL	NL	NA
Lindsay	Winter	NL	-	0.75
Lindsay	Winter	NL	-	0.75
Tottenham	Summer	NL	> 100% (2)	0.40
Tottenham	Summer	NL	> 100% (1)	0.40
Tottenham	Winter	NL	-	0.20
Tottenham	Winter	NL	-	0.20
Colborne	Summer	> 100%	> 100%	0.15
Colborne	Winter	NL	-	0.30
Colborne	Winter	NL	-	0.30
Sutton	Summer	NL	> 100% (1)	NA
Sutton	Winter	NL	-	5.15
Sutton	Winter	NL	-	5.15

( ) Numbers indicate mortality in undiluted sample

NA - Not available

lethal to either test species.

3.2.3.6 Operational Problems

Based on discussions with operating staff at the eight Sutton Process plants, some common operational difficulties at these facilities were identified, including the following:

i) Lack of preliminary treatment (screening and grit removal) caused grit, fibrous material and plastics to accumulate in the aeration basins and on the surface of the clarifier. As well as an aesthetic problem, this caused mechanical problems with aeration hardware in several plants. One plant experienced problems with clogging of return sludge pumps.

ii) Both plants which used floating aspirating mechanical aerators reported frequent mechanical failures with these units, primarily with the shafts and bearings. One plant had implemented a flushing water system to reduce the frequency of failure on these units.

iii) Most plants had freezing problems with the secondary clarifiers. Ice accumulations caused failure of the surface skimmer at one of the two plants which were equipped with this hardware.

iv) Floating scum and floating sludge were apparent on several of the secondary clarifiers. This may be related to the age of the biomass or denitrification in the clarifier or both. The lack of surface skimmers on the clarifiers make it difficult to handle the floating sludge and scum.

v) Since all plants with the exception of Stayner had only one secondary clarifier, the extended aeration plant had to be bypassed to allow maintenance to be conducted on the clarifier.

3.2.5 Sludge Quantity and Quality

3.2.5.1 Sludge Quantity

For the five facilities in operation at the time of this review, the daily sludge production discharged into the lagoon was estimated based on BOD removal efficiency and typical yields of 0.65 kg TSS per kg BOD_5 removed. The total accumulation of sludge in the lagoon since start-up as a Sutton Process plant was also estimated. These data were included in Table 3.5. In addition, the sludge piles in the lagoons at the Sutton, Colborne and Tottenham WPCPs were surveyed to estimate their size. Figures showing the sludge pile contours are included in Appendix 1.

The Sutton WPCP has the largest accumulation of sludge since it is the longest operating and one of the larger Sutton Process plants. It was estimated that about 650 tonnes of solids have been deposited in the lagoon since start-up in 1981. This represents a sludge mass of about 7 kg per m^3 of lagoon volume. At Colborne, the sludge accumulation amounts to about 5 kg per m^3 of lagoon volume. At Tottenham, the accumulation is about 2.5 kg per m^3 of lagoon. At all three facilities, the sludge mappings suggest that less than five percent of the lagoon volumes are occupied by sludge.

3.2.5.2 Sludge Quality

Samples were collected within the sludge piles at the Tottenham, Sutton and Colborne WPCPs and analyzed for nitrogen and metals to determine the suitability of these sludges for land disposal. In Table 3.11, the metal content of the sludges is compared to the land utilization guidelines (MOE, 1986) for both aerobic and anaerobically digested sludges. It should be noted that none of the Sutton Process plants sampled reported a significant input of metals from tributary industrial sources.

Sludges from all locations at all plants sampled meet the land utilization guidelines for aerobic sludges which are based on the metal content per unit mass of dry solids (mg

Table 3.11
Metal Content of Sutton Process Sludges

Metal	Guidelines for Aerobic Sludges (mg/kg solids)	Colborne WPCP		Sutton WPCP		Tottenham WPCP		Guidelines for Anaerobic Digested Sewage Sludges (Min N/Me ratio)	Colborne WPCP		Sutton WPCP		Tottenham WPCP	
		Sample		Sample		Sample			Sample		Sample		Sample	
		1	2	1	2	1	2		1	2	1	2	1	2
Arsenic	170	9	10	5	13	7	4	100	171	515	258	111	297	**84**
Cadmium	34	4	4	4	9	4	2	500	**393**	1374	**347**	**164**	**450**	**157**
Cobalt	340	2	2	1	2	6	4	50	787	3434	173	822	313	92
Chromium	2800	743	741	55	123	66	25	6	**2**	7	25	12	29	11
Copper	1700	438	445	380	875	545	266	10	**4**	12	**4**	**2**	**4**	**1**
Mercury	11	3	3	1	1	1	1	1500	**562**	1585	2475	**1174**	3507	**697**
Molybdenum	94	34	20	7	15	13	8	180	**46**	254	204	**97**	**150**	**44**
Nickel	420	232	286	18	40	29	16	40	**7**	**18**	78	**36**	66	**22**
Lead	1100	147	25	102	232	144	72	15	**11**	33	**13**	**6**	**13**	**5**
Selenium	34	4	4	4	8	6	3	500	**437**	1374	**403**	**191**	**351**	**122**
Zinc	4200	1349	1396	516	1284	622	314	4	**1**	**3.7**	**3**	**1**	**3**	**1**

Bold indicates that guidelines are not met.

metal per kg solids). Most metals exceed the land utilization guidelines for anaerobically digested sludges which are based on a minimum nitrogen-to-metal ratio. The guideline is exceeded in at least one of the six samples collected for arsenic, cadmium, chromium, copper, mercury, molybdenum, nickel, lead, selenium and zinc. Only the cobalt-to-nitrogen ratio exceeds the minimum specified in the guidelines. The exceedances of the anaerobic sludge utilization guidelines relate to the low nitrogen content of the sludge rather than to elevated metal concentration. Since these sludges are not "anaerobically digested", there is less hydrolysis of proteinaceous matter to ammonia than would be typical of an anaerobically digested sludge. The ammonia content of Sutton polishing pond sludges was in the range from about 40 mg/L to about 200 mg/L compared to typical concentrations of 250 to 1200 mg/L in anaerobically digested secondary treatment plant sludges (Environment Canada, 1984).

3.2.5.3 Effects of Sludge on Polishing Pond Effluent Quality

Liquid samples from within the sludge piles at the Sutton, Colborne and Tottenham WPCPs were collected and analyzed to determine if solubilization of ammonia was contributing to the elevated pond ammonia concentrations identified at these plants. The results indicate that within the core of the sludge pile, ammonia concentrations of 200 to 235 mg/l NH_3-N were measured at the Sutton plant, concentrations of 40 to 175 mg/L were measured at the Colborne plant and concentrations as high as 350 mg/L were measured at the Tottenham plant. These data suggest that ammonia is being released from the biological solids to the liquid phase within the sludge deposits since ammonia concentrations were typically higher in the most dense part of the sludge deposit, reducing towards the edges of the deposit and the top of the deposit.

Soluble phosphorus was measured within the sludge pile at one location at the Sutton WPCP. The result (< 0.1 mg/l P) did not indicate solubilization of phosphorus from the sludge deposit. However, the lagoon at Sutton had been batch-treated with alum about two months before the liquid sample from the sludge pile was collected. This chemical

treatment may have affected the level of soluble phosphorus present within the sludge deposit.

3.2.6 Capital and Operating Costs

Capital and operating costs for the eight plants using the Sutton Process have been compiled. The following presents a discussion of the variations in design parameters which directly affect cost, as well as various graphs presenting cost comparisons. All costs are in 1992 dollars, with historical costs being brought to present value using an ENR Index of 6537 (March, 1992). Costs were obtained either from the design consultant or plant operations staff. Breakdowns of costs, if available, have been included in Appendix 4.

3.2.6.1 Capital Costs

Capital costs for the eight Sutton Process facilities are as follows (design capacities have been indicated in brackets beside each):

- Dutton (558 m^3/d) - $2,655,800
- Rodney (590 m^3/d) - $2,655,800
- Cookstown (825 m^3/d) - $2,098,200 (includes $600,000 for additional lagoon construction)
- Colborne (1,375 m^3/d) - $2,010,700
- Stayner (1, 875 m^3/d) - $6,965,100 (includes $2,400,000 for additional lagoon construction)
- Sutton (2,046 m^3/d) - N/A
- Tottenham (2,257 m^3/d) - $1,718,200
- Lindsay (15,870 m^3/d) - $1,987,400

The costs of Rodney and Dutton are identical because they are virtually the same plant. The design for Rodney was purchased for use at Dutton after Rodney was constructed. The relatively high cost of the Stayner WPCP is due primarily to the lagoon construction at the time the extended aeration plant was constructed (two lagoons were added).

These costs have been indicated respectively in Figures 3.7 and 3.8 as normalized capital costs and normalized amortized capital costs. The amortized costs are spread over a 20 year design lifespan of the facility, using an assumed interest rate of 8%. Note that costs for the Sutton WPCP were unavailable (the design consultant was unable to provide this information due to the fact that the plant was designed over 15 years ago). Detailed capital cost breakdowns have been included in Appendix 4.

Upon reviewing Figures 3.7 and 3.8, a definite trend exists in that the smaller plants exhibit a higher capital cost than do the large ones. Rodney and Dutton are abnormally high due to conservativeness of design. For instance, the Lindsay WPCP has such a low treatment cost due to the fact that it was a retrofit facility, with an existing earthen lagoon with floating surface aerators for aeration and a rather inexpensive control building. In contrast, the Rodney and Dutton plants used a series of reinforced concrete aeration tanks, bridge mounted mechanical surface aerators, and had a spacious control building complete with laboratory facilities, office, washrooms, housing for return sludge pumps and chemical feed pumps, and storage area. The engineering costs were also very high on Rodney, since the plant was originally designed as a full tertiary plant, then modified to a Sutton Process plant due to cost considerations.

Aside from specific design criteria considerations, the scope of work is not identical for each plant constructed. The plants at Cookstown and Stayner involved construction in whole or in part of the polishing lagoons, while the other plants were strictly retrofit facilities.

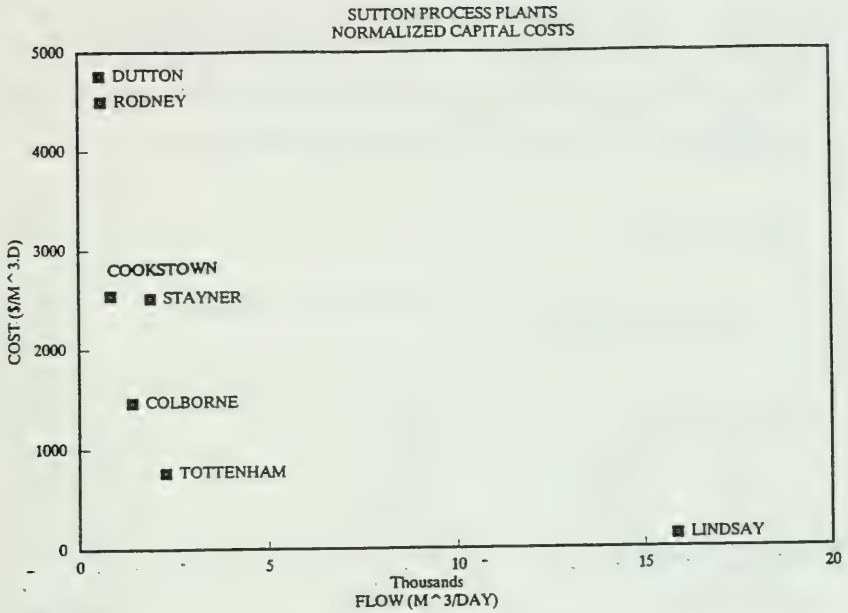


FIGURE 3.7

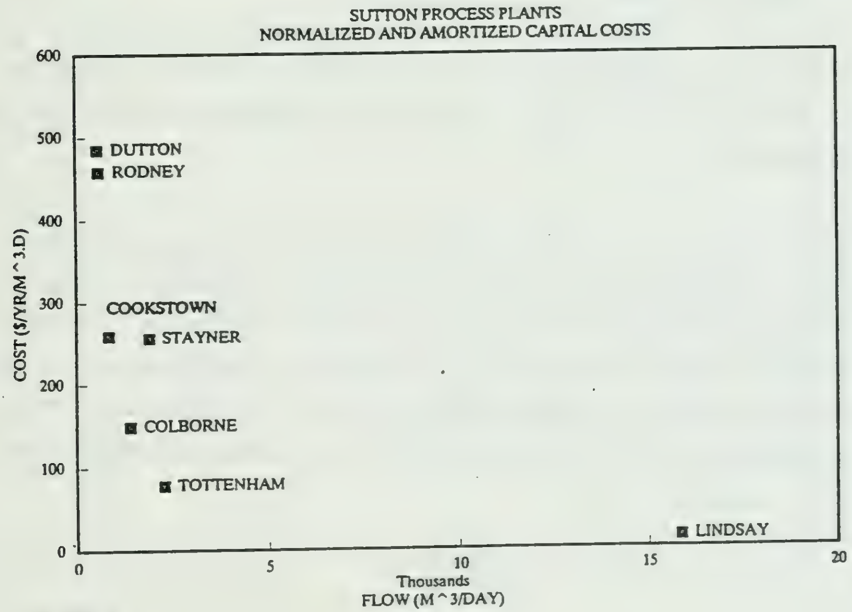


FIGURE 3.8

Figure 3.9 indicates the variation in plant cost with the year of construction. The trend seems to indicate that the newer facilities have become more expensive over recent years. This is likely attributable to differences in design approaches (for example, more concrete structures in the new facilities, more conservative design, etc).

3.2.6.2 Operation Costs

Yearly operation costs for the Sutton Process facilities are as follows:

- Dutton (558 m^3/d) - N/A
- Rodney (590 m^3/d) - N/A
- Cookstown (825 m^3/d) - \$ 67,500
- Colborne (1,375 m^3/d) - \$ 84,700
- Stayner (1,875 m^3/d) - N/A
- Sutton (2,046 m^3/d) - \$112,500
- Tottenham (2,257 m^3/d) - \$ 77,200
- Lindsay (15,870 m^3/d) - \$283,000

The costs for Dutton, Rodney and Stayner are not available, since they only began operating in 1991. Detailed breakdowns of operation and maintenance costs have been included in Appendix 4.

Figure 3.10 indicates capital and operation/maintenance costs for four (4) of the Sutton facilities. These costs are yearly lifecycle costs of the facility over its 20 year design life, and range from \$340/m^3.d (\$1,540,000/MGD) of design flow at Colborne, to \$31/m^3.d (\$140,000/MGD) of design flow at Lindsay. This range is in part attributable to the economy of scale of large versus small facilities, as well as the observation that the smallest facilities (Rodney WPCP and Dutton WPCP) were also the most conservatively designed.

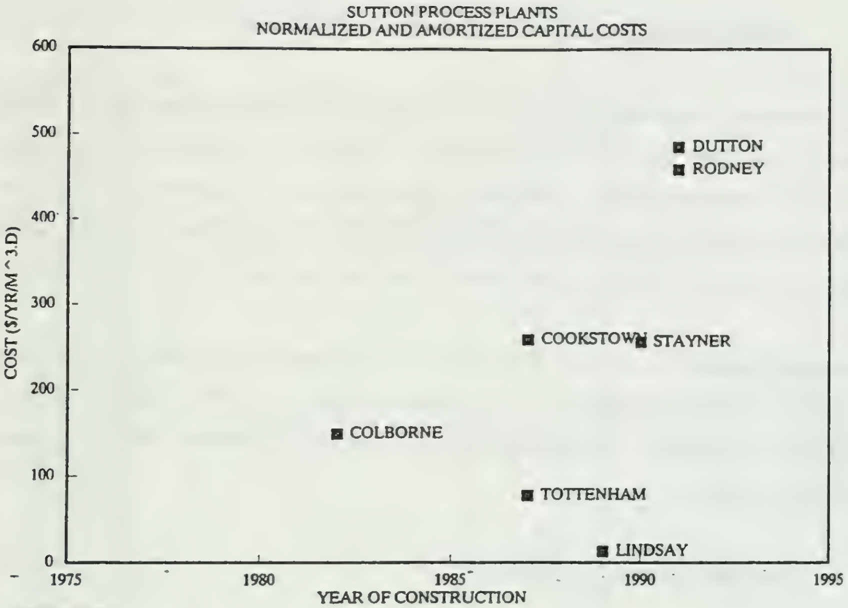


FIGURE 3.9

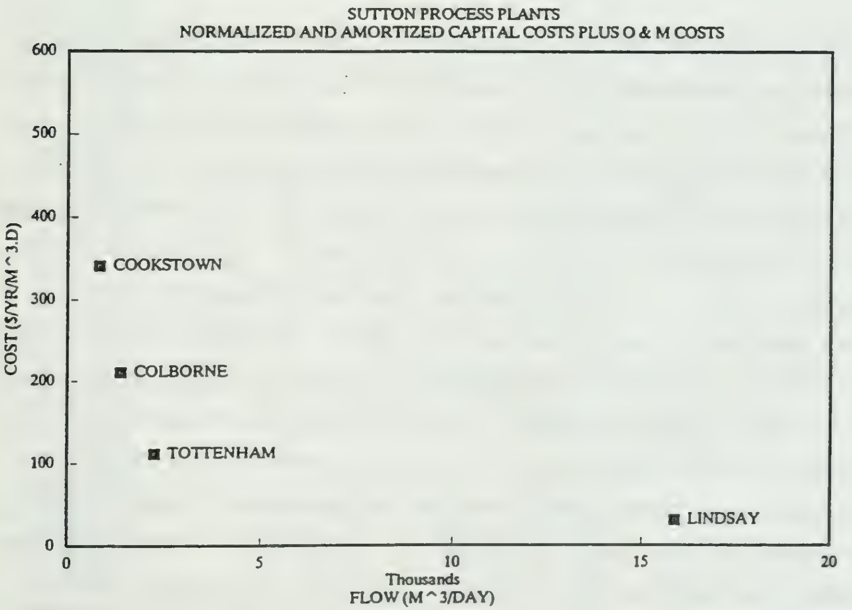


FIGURE 3.10

3.2.6.3 Annual Capital and Operation & Maintenance Costs

Table 3.12 indicates the breakdown of amortized capital cost and yearly operation and maintenance costs over the life of each facility. As can be seen, because the Lindsay WPCP had the lowest amortized capital cost, it therefore had the highest percentage of the total cost attributable to operations and maintenance. It should also be pointed out that Lindsay also has the lowest operation and maintenance cost (in $\$/m^3.d$ treated).

This low annual cost for the Lindsay WPCP is not only because of its size, but also due to the relatively unsophisticated design. For example, the Lindsay WPCP employs a large aerated lagoon with floating surface aerators, and has little concrete structure when compared to other plants.

3.3 The New Hamburg Process

3.3.1 Process Description

In the "New Hamburg Process", the lagoon effluent is applied to the surface of a granular media filter to upgrade the final effluent quality prior to discharge. As the lagoon effluent passes through the filter, particulate matter including algal cells are strained from the wastewater and accumulate in the upper layers of the sand media. Depending on operating conditions, nitrification can also occur in the filter as the wastewater is exposed to nitrifying micro-organisms attached to the filter media. The process is shown schematically in Figure 3.11. The multi-cell filters are operated on a periodic or intermittent basis and flow through the filters is by gravity. A distribution system is used to flood the surface of the filters and underdrains provided to collect the filtered final effluent. No filter backwash is provided. The filter is loaded until the surface becomes plugged with particulate matter removed from the lagoon effluent. When this occurs, the filter is taken off line for cleaning, maintenance or self-regeneration. The intermittent sand filtration process, or "New Hamburg Process" as it is referred to in Ontario, has been widely used in the United States for upgrading lagoon effluents. Detailed discussion of

Table 3.12

COMPARISION OF AMORTIZED CAPITAL COSTS TO 0 & M COSTS FOR SUTTON PLANTS

	Amortized Capital Cost ($/year/m^3.d)	O & M Cost ($/year/m^3.d)	Total ($/year/m^3.d)	% Amortized Capital Cost	% O & M Cost
Dutton	485	N/A			
Rodney	458	N/A			
Cookstown	259	82	341	76%	24%
Colborne	149	62	211	71%	29%
Stayner	378	N/A			
Sutton	N/A	55			
Tottenham	78	34	112	70%	30%
Lindsay	13	18	31	42%	58%

NOTE: ENR Index = 6537
Interest Rate = 8%

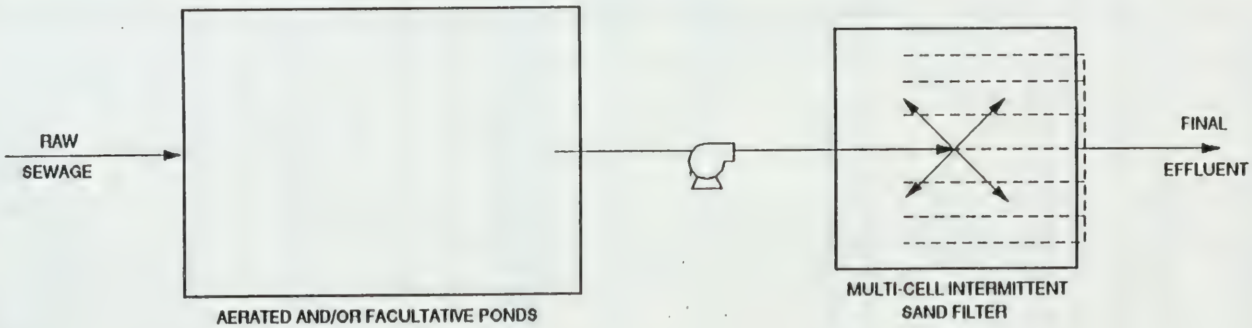


Figure 3.11 Schematic of New Hamburg Process

U.S. applications of the process are contained in Appendix 3.

Experience in the U.S. has shown that the duration of a filter run depends on the suspended solids content of the lagoon effluent, on the hydraulic loading on the filter and on the particle size of the filter media. In the Southern U.S. where climatic conditions are conducive to algal growth most of the year, short filter runs are often experienced. Night time operation of the filters has been found to extend the length of the filter runs. Maintenance of the clogged filter bed typically involves removal and replacement of the upper layers of the filter bed.

3.3.2 Design of New Hamburg Process Plants

As identified in Table 3.2, there are two examples of the New Hamburg Process in Ontario currently. The original application of the technology has been operational at New Hamburg since 1980. This represented an upgrade from an existing facultative lagoon system which had been operational at the site since the 1960's. A completely new facility utilizing this technology was installed at Schomberg in 1991. Table 3.13 summarizes the key process design parameters for these two facilities. More detailed information on each plant is included in Appendix 2.

The design data summarized in Table 3.13 reflect differences in the effluent quality requirements for the two plants. The limits on these facilities as stated in the Certificates of Approval are summarized in Table 3.14. The limits at the Schomberg WPCP are substantially more restrictive in terms of BOD_5, phosphorus and ammonia than those at the New Hamburg WPCP. In the summer months (July through September), no discharge from the Schomberg WPCP to the receiving stream is allowed. In June and October, the effluent quality limits are 6 mg/L BOD_5 and 1.0 mg/L NH_3-N.

TABLE 3.13
SUMMARY OF DESIGN CRITERIA
NEW HAMBURG PROCESS PLANTS

PLANT	NEW HAMBURG	SCHOMBERG
DESIGN CAPACITY (m ^ 3/d)	2 700	683 2595 (peak)
YEAR OF START-UP	1981	1991
DESIGN :		
PRELIM. TREATMENT		
SCREENING	NO	NO
GRIT REMOVAL	NO	NO
AERATED CELL		N/A
DESIGN HRT (d)	4.2	
BOD LOADING (kg/m ^ 3d)	0.05	
(kg/m ^ 2d)	0.10	
AERATION PROCESS	3 @ 18.6 kW blowers 52 MAT aerators	
AIR FLOW (L/min per m ^ 3)	2.20	
FACULTATIVE CELLS		
NUMBER	TWO	THREE
SURFACE AREA (ha)	11.1	8.8
RETENTION TIME (d)	102	180
FILTER		
NUMBER	FOUR	THREE
SURFACE LOADING (L/m ^ 2 d) *	562.5	94.9
PHOSPHORUS REMOVAL		
CHEMICAL	ALUM	ALUM
ADDITION POINT	effluent of aerated cell	lagoons or filter

* Based on design flow/total surface area

TABLE 3.14

SUMMARY OF NEW HAMBURG PROCESS PLANTS

EFFLUENT QUALITY PARAMETERS

PLANT	NEW HAMBURG			SCHOMBERG		
DESIGN CAPACITY (m ^3/d)			2,700			683
YEAR OF START-UP			1981			1991
Effluent Limits:						
BOD (mg/L)	monthly	May-Oct	15.00	monthly	Apr	20.00
		Nov-Apr	30.00		May	10.00
					Jun	6.00
					Oct	6.00
					Nov	10.00
TSS (mg/L)					monthly	15.00
TP (mg/L)		annual	1.00		monthly	0.30
LOADING (kg/yr)					annual	75.00
TKN (mg/L)	Jan	@ 5 Cels.	20.00			
	Feb	@ 5 Cels.	25.00			
	Mar	@ 10 Cels.	25.00			
	Apr	@ 15 Cels.	20.00			
	May	@ 20 Cels.	10.00			
	Jun	@ 25 Cels.	10.00			
	Jul	@ 25 Cels.	10.00			
	Aug	@ 25 Cels.	10.00			
	Sep	@ 20 Cels.	10.00			
	Oct	@ 15 Cels.	10.00			
	Nov	@ 10 Cels.	15.00			
	Dec	@ 5 Cels.	15.00			
T NH4 (mg/L)	Jan	@ 5 Cels.	15.00	monthly	Apr	8.00
	Feb	@ 5 Cels.	20.00		May	2.50
	Mar	@ 10 Cels.	20.00		Jun	1.00
	Apr	@ 15 Cels.	15.00		Oct	1.00
	May	@ 20 Cels.	5.00		Nov	3.50
	Jun	@ 25 Cels.	5.00			
	Jul	@ 25 Cels.	5.00			
	Aug	@ 25 Cels.	5.00			
	Sep	@ 20 Cels.	5.00			
	Oct	@ 15 Cels.	5.00			
	Nov	@ 10 Cels.	10.00			
	Dec	@ 5 Cels.	10.00			
OTHER	C of A limits capacity to 2,300 m3/d			Discharge flow limited to 683 m ^3/d except in April (2,048 m ^3/d) and May (1,368 m ^3/d).		

3.3.2.1 Lagoon Design

Generally, intermittent slow sand filters will be added to an existing lagoon system to upgrade effluent quality and/or to increase hydraulic capacity. Under these circumstances, the lagoon system design reflects the original plant design more than a design specific for the New Hamburg Process. The retention time of the existing lagoon is generally determined by storage requirements (annual discharge, seasonal discharge or continuous discharge) and follows conventional design practice for waste stabilization ponds. Additional storage capacity may need to be provided in the case of an existing continuous discharge lagoon to allow seasonal operation of the downstream filter.

The New Hamburg WPCP incorporates a 4.2 day HRT aerated lagoon upstream of two facultative ponds of 11.1 ha which provide a total storage time of 102 days at design flow. Aeration to the aerated lagoon is provided by submerged MAT aerators supplied from three 18.6 kW blowers. The design organic loading on the aerated cell is approximately 0.05 $kg/m^3.d$. The aerated cell was added at the time that the original facultative ponds were upgraded in 1980. At that time, the facility received organic loads from a local cheese plant. The aerated lagoon was intended to reduce the organic loading on the existing facultative ponds. One of the facultative ponds was deepened at the time of the upgrading from the original depth of 2.3 m to about 3.0 m to provide additional storage capacity.

The Schomberg WPCP was constructed as a completely new facility and incorporates three facultative ponds with a total surface area of 8.8 ha providing a nominal retention time of 180 days. The organic loading on the lagoons at design flow is 15.3 kg BOD_5/ha.d, slightly lower than the maximum of 22 kg/ha.d suggested by MOE Guidelines (MOE, 1984) for facultative waste stabilization ponds. Pond sizing was based on providing six months of storage capacity because of the discharge restrictions on the plant.

3.3.2.2 Phosphorus Removal

Both the New Hamburg and Schomberg WPCP add alum to achieve phosphorus removal. At the New Hamburg plant, alum is added at the discharge of the aerated lagoon upstream of the first facultative pond. At Schomberg, the alum feed system is designed to allow alum addition to the raw sewage entering the facultative ponds and/or to the filter feed. The operating authority are also considering batch treatment of the lagoons prior to feeding the filters if elevated phosphorus concentrations in the lagoon are detected.

3.3.2.3 Filter Design

The New Hamburg WPCP has four filter cells, each 0.12 ha in area for a total filtration area of 0.48 ha. This is equivalent to a filtration area of 1.8 m^2 per m^3/d of treatment capacity or a nominal loading of about 560 L/d per m^2 of filter. This design hydraulic loading is consistent with that typically used in intermittent sand filter installations in the U.S., normally designed for about 480 $L/m^2.d$. At New Hamburg, two filters are operated at a time and the instantaneous hydraulic loading on the filters is about 3.2 $m^3/m^2.d$, based on the filter feed pump output of 90 L/s (one pump operating).

The sand filters at the Schomberg WPCP are considerably larger per unit of design capacity than those at the New Hamburg WPCP. This reflects the discharge restrictions during summer months and the operating philosophy which is based on maximizing filter throughput during the spring when effluent limits are least restrictive. Total filter surface area is a function of the plant size as well as the allowable operating period which may be influenced by climatic conditions or receiving stream considerations. At the Schomberg WPCP, three filter cells are provided, each 0.24 ha in area for a total filtration area of 0.72 ha. This is equivalent to a filtration area of 10.5 m^2 per m^3 of treatment capacity or a nominal loading of about 95 L/d per m^2 of filter, less than 20 percent of the nominal loading on the New Hamburg filters. However, the instantaneous filter loading, based on one filter in operation and a maximum filter feed pump output of 126 L/s, is

about 2.8 $m^3/m^2.d$ (one feed pump operating). This instantaneous loading is comparable to that at the New Hamburg plant.

3.3.3 Operation and Performance of the New Hamburg Process

The Schomberg WPCP was commissioned in October of 1989. During the first year and a half of operation, the lagoons were being filled. There was no discharge from the plant until 1991 when the filters were first operated. At the time of the site visit, only one filter cell had been operated in the spring of 1991. Therefore, the only representative data on which to base an analysis of the performance of the New Hamburg Process are data from the New Hamburg WPCP.The recent operating and performance history (1990/91) of the New Hamburg WPCP is summarized in Table 3.15.

More detailed data are presented in Appendix 2.

3.3.3.1 Operating Conditions

The New Hamburg WPCP is currently operating at about 60 percent of its design capacity. Hence, retention times in the aerated and facultative lagoons average about 7 days and 165 days, respectively. Organic loadings on the aerated cell and the facultative ponds are relatively low compared to conventional design practices. Between 70 and 80 percent BOD_5 removal has been achieved in the aerated lagoon, reducing the loading on the facultative ponds to about 5 kg/ha.d or 25 percent of the maximum suggested by MOE Guidelines (MOE, 1984).

The facultative ponds provide about 165 days of storage based on average day flows. The intermittent sand filters at the plant are normally started up as early in March as possible after the filter surface is ice-free. Filter operation stops in about mid-December depending on weather conditions. Hence, there is a period of about 90 days when the filters cannot operate in the winter months. The collection system servicing the New Hamburg WPCP is subject to significant infiltration and inflow under wet weather

TABLE 3.15 CURRENT OPERATING CONDITIONS NEW HAMBURG PROCESS PLANT		
PLANT	NEW HAMBURG	
YEAR	1990	1991 (Jan–Aug)
AVERAGE DAY FLOW (m3)	1676	1673
MAXIMUM DAY FLOW (m3)	4530	3990
RAW SEWAGE:		
AVG. BOD INF. (mg/L)	186.0	120.5
AVG. TSS INF. (mg/L)	314.8	177.4
AVG. TKN INF. (mg/L)	45.4	44.3
AVG. TP INF. (mg/L)	9.3	9.5
AERATED CELL		
OPERATING HRT (d)	6.9	6.9
BOD LOADING (kg/m ^ 3d)	0.03	0.02
AERATED CELL EFFLUENT:		
AVG. BOD (mg/L)	34.0	36.4
STD.DEV.	11.6	10.2
AVG. TSS (mg/L)	43.6	44.0
STD.DEV.	19.1	26.7
AVG. TP (mg/L)	5.8	4.8
STD.DEV.	1.1	1.2
FACULTATIVE LAGOONS		
OPERATING HRT (d)	165.0	165.0
AVERAGE BOD LOADING (kg/1000m ^ 2*d)	0.51	0.55
CELL 2 EFFLUENT:		
AVG. BOD (mg/L)	12.5	11.5
STD.DEV.	6.3	2.0
AVG. TSS (mg/L)	15.8	18.0
STD.DEV.	9.0	9.5
AVG. TKN (mg/L)	19.0	17.6
STD.DEV.	4.0	3.9
AVG. NH3–N (mg/L)	15.5	14.3
STD.DEV.	4.4	4.5
AVG. NO(T)–N (mg/L)	1.1	0.8
STD.DEV.	0.9	0.5
AVG. TP (mg/L)	1.2	0.7
STD.DEV.	0.5	0.2
FILTER		
ANNUAL SURFACE LOADING (m ^ 3/m ^ 2)	195	153
INSTANTANEOUS SURFACE LOADING (L/m ^ 2*d)	3240	3240
FILTER EFFLUENT:	Mar – Dec	Mar – Aug
AVG. BOD (mg/L)	2.2	1.8
STD.DEV.	2.8	0.8
AVG. TSS (mg/L)	1.7	1.1
STD.DEV.	1.3	0.9
AVG. TKN (mg/L)	2.0	1.1
STD.DEV.	2.3	0.7
AVG. NH3–N (mg/L)	1.2	0.6
STD.DEV.	1.9	0.7
AVG. NO(T)–N (mg/L)	6.9	8.6
STD.DEV.	3.9	7.7
AVG. TP (mg/L)	0.5	0.4
STD.DEV.	0.1	0.1

conditions. Therefore, storage volume in the facultative ponds can be limited due to high flows in the spring months. Occasionally, it has been necessary to discharge directly from the lagoons in the spring if there is no additional storage available and the filters have not thawed sufficiently to permit their operation.

Normally, two filter cells are in operation and two are on stand-by. Discharge flow to the receiving stream must be proportioned to streamflow. Hence, the filter operating hours are varied from as little as two hours to as much as 8 hours per day. Filter feed pumps are timer operated to allow the operator to preset a filter feed sequence consistent with the streamflow limitations. The filters are operated at night and allowed to drain in the daytime hours. Each filter is typically loaded for about 20 to 30 days depending on the suspended solids content of the lagoon effluent before it is taken off line to dry. Drainage time after a feeding cycle is used to determine the need to switch a particular filter to the standby mode. Filters are operated continuously from the start-up in the spring until the freeze-up in the fall unless an algal bloom occurs which would produce short filter run times. In such cases if storage volume is available in the lagoons, filter operation will be suspended until a better quality lagoon effluent is available to apply to the filters.

Media has never been removed and disposed of from the filter beds as is typical in U.S. facilities as part of the filter maintenance cycle. Initially, filter surface maintenance at the New Hamburg WPCP involved mechanical surface cultivation to remove surface vegetation. This practice was discontinued because it seemed to contribute to high effluent phosphorus concentrations.

Manual removal of vegetation was found to be too time-consuming. The current practice is to burn off the surface vegetation annually. There has not appeared to be a reduction of filter throughput which could be related to clogging of the filter surface with vegetation or trapped particulate matter.

3.3.3.2 Plant Performance

The average quality of the final facultative cell (cell 2) which feeds the filters and the filter effluent are presented in Table 3.15 for 1990 and the first half of 1991. Contaminant concentrations in the facultative pond are consistent with those achieved in other Southern Ontario fill-and-draw lagoons (refer to Section 2.0). BOD_5 and TSS are reduced to the 15 mg/L range and total phosphorus to about 1 mg/L. Ammonia concentrations average about 15 mg/L on an annual basis but show the seasonal effects typical of facultative lagoon systems.

The intermittent sand filters produce a tertiary quality effluent. On average, BOD_5 and TSS is reduced to less than 5 mg/L and total phosphorus to less than 1.0 mg/L. Ammonia nitrogen levels averaged 1.2 mg/L NH_3-N in 1990. The elevated concentrations of oxidized nitrogen (nitrate plus nitrite) suggest that a large fraction of the ammonia removal in the filters is associated with nitrification. H_2S has not been measured at detectable levels in the filter effluent.

MOE undertook comprehensive monitoring of the New Hamburg WPCP in 1991 and 1992. Lagoon effluent (filter influent) and filter effluent ammonia concentrations are compared in Figure 3.12 from the start-up of filter operation on March 5 until mid July of 1991. Similar data for nitrate concentrations are presented in Figure 3.13. These data suggest that nitrification in the filter begins very quickly after start-up despite about three months of inactivity under winter conditions. Based on the limited sampling data, filter effluent NH_3-N did not exceed about 4.0 mg/L during start-up despite influent concentrations of about 13 mg/L NH_3-N. Oxidized nitrogen was present immediately after start-up in the filter effluent indicating that nitrification was occurring. The first two filters were operated for a period of about six weeks (April 17) before they were taken off-line and the other two filters put into operation. There was a minor increase in the effluent NH_3-N concentration (and a related decrease in effluent NO_3-N) at that time as shown in Figures 3.12 and 3.13. However, efficient nitrification was rapidly re-established. The declining NH_3-N concentration in the lagoon with the onset on warmer conditions in May

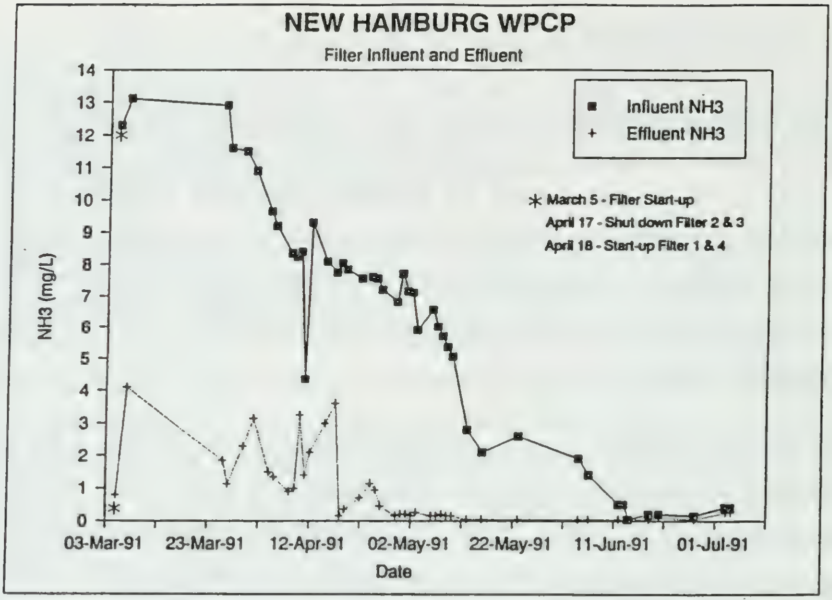


Figure 3.12

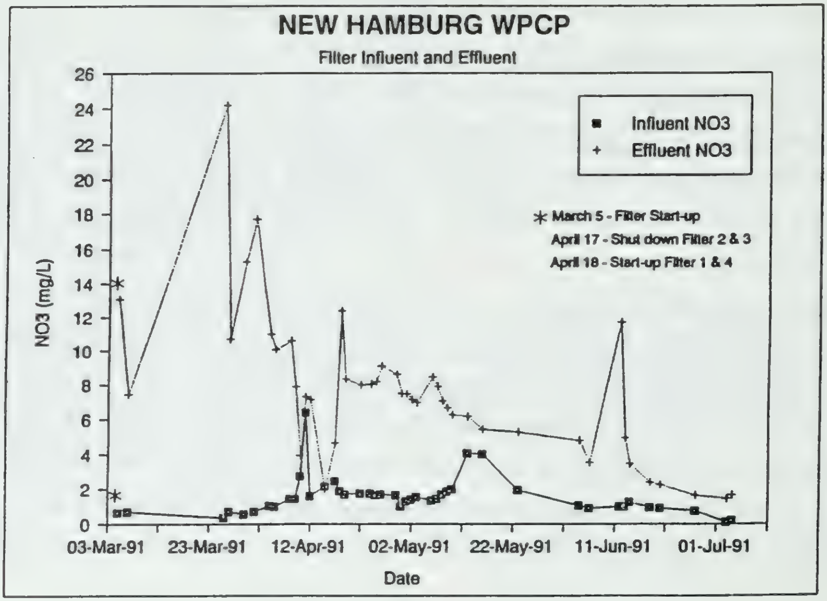


Figure 3.13

and June is also apparent in Figure 3.12.

Unfortunately, the 1991 monitoring data were not continuous during the filter start-up period between about March 6 and the end of March. Therefore, limited data were available immediately after start-up. To provide additional information regarding the start-up of nitrification in the filter, additional sampling was done in 1992. In this case, the filter effluent was sampled hourly for the first four hours of filter operation on March 9 and after 24 hours of operation. A composite sample of the filter effluent for 20 hours (4 hours to 24 hours) was collected as well as a 24 hour composite sample representative of the first full day of filter influent (lagoon effluent). These results as summarized in Table 3.16 also suggest a rapid initiation of nitrification in the filter, although a longer period of effluent sampling would be required to confirm this conclusion.

3.3.3.3 Effluent Toxicity

Samples of the lagoon effluent (filter influent) and the filter effluent were collected from the New Hamburg WPCP in March 1991 shortly after start-up of the filter and tested for acute toxicity using rainbow trout and Daphnia magna as the test species. The results of this limited toxicity testing is presented in Table 3.17. The influent to the filter had an LC_{50} of 90.1 percent based on rainbow trout. Influent testing with Daphnia magna was not done. The filter effluent was non-lethal to Daphnia magna and had an LC_{50} greater than 100 percent based on the rainbow trout test.

Lagoon effluent samples tend to have elevated pH levels, particularly during the summer months when there is considerable photosynthetic activity. In 1991 at the New Hamburg WPCP, the lagoon effluent pH ranged from about 8.1 at start-up to 8.9 in mid-July. Despite active nitrification in the filter, the pH reduction across the filter was minimal. The minimum pH of the filter effluent was about 7.7 and the maximum in mid-July was about 8.4. The alkaline pH of the filter effluent will increase the toxic effects of ammonia since more of the ammonia-nitrogen will be present in the more toxic unionized ammonia (NH_3) form. For example, to achieve an unionized ammonia concentration of 0.1 mg/L - the

Table 3.16 Filter Performance During Start-Up (March 09/92)			
Sample	Concentration (mg/L)		
	NH_3-N	NO_3-N	NO_2-N
Filter Influent (24-hr. comp.)	18.10	< 0.20	0.035
Filter Effluent			
0 hour	0.30	36.00	0.205
1 hour	0.40	12.40	0.085
2 hour	1.55	3.90	0.070
3 hour	5.45	2.90	0.105
24 hour	6.80	14.40	1.100
(20 hr comp.)	6.60	9.75	0.720

Note: Lagoon Temperature 1.0 °C
Filter Effluent Temperature 3.0 °C
Air Temperature 7.0 °C.

Table 3.17 New Hamburg Process		
Toxicity Test Results		
Location	**Trout LC_{50}**	**Daphnia Magna LC_{50}**
New Hamburg (Filter Influent-Lagoon Effluent), March 25, 1991	NL	-
New Hamburg (Filter Effluent), March 25, 1991	> 100% (1)	NL

( ) Numbers indicate mortality in undiluted sample.

NL Non-lethal

level typically accepted as non-lethal (Beak 1991) - at a pH of 8.5 and 10°C would require that the total ammonia concentration be maintained at less than 1.8 mg/L. The pH reduction which occurs across the filter will depend on the alkalinity content of the wastewater. Data for intermittent sand filtration plants polishing lagoon effluents in Georgia and California which are presented in Appendix 3 of this report shows pH reductions from levels of 9.3 to 9.5 in the lagoon effluents to 7.0 to 7.3 in the filter effluents.

3.3.3.4 Operating Problems

Significant operating problems with the filters at New Hamburg were encountered when the plant was commissioned until an effective operating strategy was developed. The filters were originally designed to operate on a year-round basis. Freezing problems eliminated this as an effective operating strategy. During the initial period of operation, the filters were operated in the day time. Algal growth on the flooded filter surface quickly fouled the filter media. Night time operation was found to be necessary to prevent excessive algal growth on the filters. It was also found that discontinuing filter operation during periods of high algal growth in the lagoons prevents blinding of the filter media. A variety of methods were tried to remove vegetative growth from the filters. Burning has been utilized recently as the most effective process. The New Hamburg WPCP operation now provides a tertiary quality effluent with minimal operating problems with the filter and with minimal labour (about 4 person-hours per day).

Limited storage capacity in the facultative ponds remains a problem which is compounded by high wet weather flows and an accumulation of alum sludge in the first facultative pond. This has required earlier start-up of the filters and/or discharge of lagoon effluent direct to the receiving stream in the spring. Plugging of the aerators in the aerated lagoon has also been a problem, but is not associated with the New Hamburg Process specifically.

3.3.4 Capital and Operating Costs

Capital and operating costs for both New Hamburg Process Plants are listed herein. All costs are in 1992 dollars, with historical costs being brought to present value using a March 1992 ENR Index of 6537.

3.3.4.1 Capital Costs

Capital costs for the two New Hamburg Process plants are as follows (design capacities have been indicated beside each):

- New Hamburg (2,700 m^3/d) - $2,653,200 (includes $580,000 for lagoons)
- Schomberg (683 m^3/d) - $2,027,500 (includes $830,000 for lagoons)

Figures 3.14 and 3.15 are the normalized capital cost and normalized amortized capital costs as a function of plant size (detailed breakdowns of these costs can be seen in Appendix 4). As can be seen in both of these figures, the Schomberg facility is considerably more expensive than the New Hamburg. This is due primarily to the fact that Schomberg WPCP was constructed as a "Green Field" facility, whereas New Hamburg WPCP was essentially a retrofit, with the exception of the addition of an aerated lagoon.

3.3.4.2 Operation Costs

Yearly operation costs for the New Hamburg Process facilities are as follows:

- New Hamburg - $120,000
- Schomberg - $ 47,600

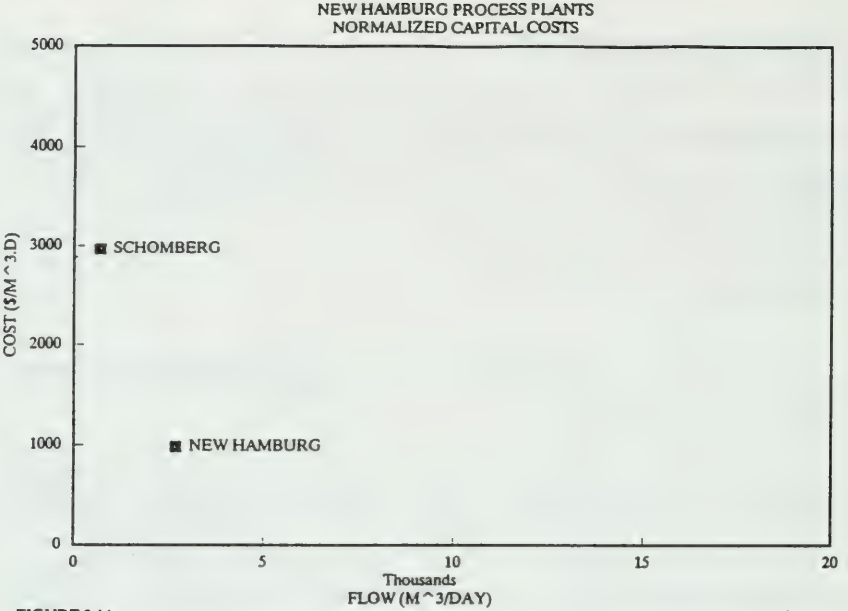


FIGURE 3.14

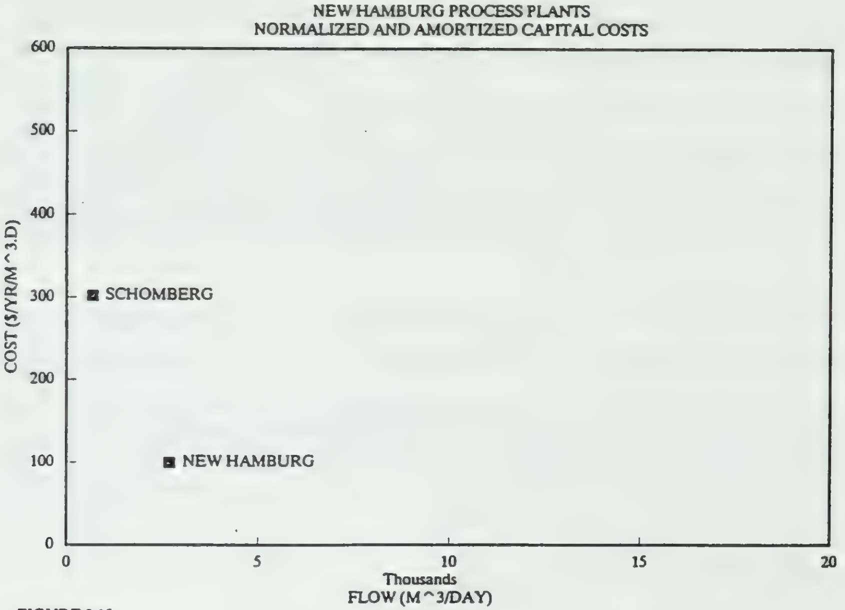


FIGURE 3.15

The differences here are likely due to two reasons. Firstly, Schomberg WPCP was still under the construction warranty period in the first months of 1991. Secondly, the New Hamburg plant has higher hydro costs, due to the operation of the aerated lagoon. A more detailed breakdown of operation costs has been included in Appendix 4. It should be noted that these operating costs do not include any costs of media replacement or sludge removal from the lagoon.

The combined capital and operation costs, normalized and amortized, for the above noted facilities have been indicated in Figure 3.16. This follows much the same trend as the previous figures.

3.3.4.3 Annual Capital and Operating & Maintenance Costs

Table 3.18 is a breakdown of yearly operation costs and amortized capital costs. As is indicated by the Table, total annual capital plus operations & maintenance costs are \$372/m^3.d (\$1,690,000/MGD) for the Schomberg WPCP and \$220/m^3.d (\$1,000,000/MGD) for the New Hamburg WPCP. The higher percentage operation and maintenance costs for the New Hamburg plant, as discussed previously, is reflected in this table.

3.4 Summary of Alternatives for Lagoon Effluent Quality Upgrading

3.4.1 Sutton Process

Based on the performance history of the three examples of Sutton Process plants which have operated for a significant period of time in Ontario, the Sutton Process produces an improvement in effluent quality compared to conventional lagoons in terms of BOD_5 and TSS concentrations. Seasonal increases in TSS concentrations characteristic of conventional lagoons do not occur in the effluents from the Sutton Process plants. Based on limited data available, H_2S production in the conventional lagoons in winter months seems to be eliminated by the Sutton Process operating mode. There are insufficient

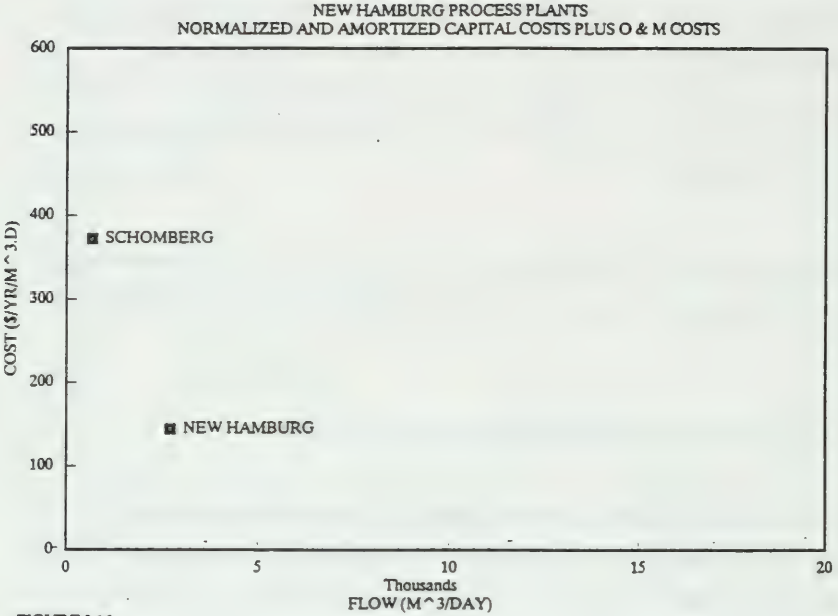

NEW HAMBURG PROCESS PLANTS
NORMALIZED AND AMORTIZED CAPITAL COSTS PLUS O & M COSTS
COST ($/YR/M ^ 3.D)
600
500
400
300
200
100
0
SCHOMBERG
NEW HAMBURG
0
5
10
15
20
Thousands
FLOW (M^3/DAY)

FIGURE 3.16

Table 3.18

COMPARISON OF AMORTIZED CAPITAL COSTS TO O & M COSTS FOR NEW HAMBURG PLANTS

	Amortized Capital Cost ($/year/m^3.d)	O & M Cost ($/year/m^3.d)	Total ($/year/m^3.d)	% Amortized Capital Cost	% O & M Cost
Schomberg	302	70	372	81%	19%
New Hamburg	100	120	220	45%	55%

NOTE: ENR Index = 6537
Interest Rate = 8%

data available on which to base a conclusion regarding the comparative toxicity of effluents from Sutton Process plants and conventional lagoons, although effluents from all Sutton Process plants in summer and winter were not acutely lethal to rainbow trout and Daphnia magna.

The performance improvement in terms of ammonia concentrations achieved in the Sutton Process compared to conventional lagoons is significant during the initial period of operation. However, ammonia is solubilized from the sludge discharged to the polishing pond. This release of ammonia results in a small but measurable increase in the ammonia concentration across the polishing pond even in the first years of Sutton Process operation. The operating time required before the increase in ammonia concentration in the pond effluent will be significant will depend on the size of the pond relative to the amount of sludge accumulated; however, the data from the Sutton and Colborne WPCP suggest that for these systems, the ammonia increase is significant after about six to seven years of operation. Performance data from the Sutton WPCP also suggests that the operation of the polishing pond has a negative effect on the phosphorus content of the pond effluent. This effect was not apparent at any of the other Sutton Process plants and the cause of the increase in phosphorus concentration at Sutton cannot be established based on available data.

One of the intended purposes of discharging sludge to the polishing pond was to achieve denitrification; however, none of the existing examples of Sutton Process plants are required to denitrify to meet specific oxidized nitrogen or total nitrogen limits. Hence, the benefits of this sludge management practice were primarily related to the cost savings associated with eliminating sludge haulage and disposal costs. This cost savings may not be realized if sludge must be removed from the lagoon and disposed after five years of operation to prevent high ammonia concentrations in the pond effluent. The relative costs of this method of handling sludge are compared to the costs of a conventional sludge disposal practice in Section 4.0. None of the existing Sutton Process plants have removed sludge from their lagoons. Hence the procedures and costs associated with sludge withdrawal are only estimates.

More recent examples of the Sutton Process including the Dutton, Rodney and Stayner WPCPs have been conservatively designed compared to the Sutton and Colborne WPCPs. Aeration basin hydraulic retention times in excess of 30 hours and secondary clarifier peak day surface overflow rates of less than 30 $m^3/m^2.d$ have been used. The performance history of the Sutton and Colborne WPCPs suggest that nitrification can be maintained year-round in the extended aeration plant at higher hydraulic and organic loadings.

3.4.2 New Hamburg Process

There are limited data on which to base an assessment of the New Hamburg Process since the New Hamburg WPCP is the only Ontario example with an operating history. Despite the limited data, the available information suggests that the use of an intermittent slow sand filter to upgrade the effluent from a conventional lagoon system results in a significant improvement in terms of BOD_5, TSS, TP, TKN, NH_3-N and H_2S concentrations. The very limited toxicity data available also suggest that a reduction in acute lethality is accomplished across the filter in the spring.

Based on the experience at New Hamburg, seasonal night-time operation of the filter is necessary. Maintenance on the filter has been minimal since this operating procedure was adopted. Because of the limitations in available operating hours, lagoon and filter sizing reflects the available operating time of the filter. The design hydraulic loading of the New Hamburg WPCP filters is about 560 $L/m^2.d$ compared to typical values of 480 $L/m^2.d$ in similar U.S. installations. The Schomberg WPCP is designed for significantly lower average loadings because of seasonal restrictions on plant discharge flows.

3.4.3 Other Upgrading Alternatives

A preliminary review of other alternatives which have been applied elsewhere in Canada or the United States for upgrading lagoon effluents was undertaken as part of this investigation. This review focused on technologies which would address seasonal TSS,

ammonia and H_2S problems in conventional lagoon systems under climatic conditions similar to those in Ontario. The technologies included in this review were:

- intermittent sand filters, an upgrading approach similar to the New Hamburg Process;

- rock filters;

- constructed wetlands with a free water surface (FWS);

- constructed wetlands with subsurface flow (SF);

- aquaculture; and,

- land application using the slow rate process (SR).

A brief summary of the key findings regarding the applicability of these processes in Ontario is presented in Table 3.19. More details regarding the design, operation, performance and cost of these technologies is presented in the full report which is included in Appendix 3. The conclusions regarding the applicability of these technologies are paraphrased below. "Snowfluent", in which lagoon effluent is used to produce artificial snow, was not considered in this review of other alternatives but has been tried in Ontario (Huber and Palmateer, 1985).

- Rock filters are unacceptable for lagoon upgrading in Ontario due to their marginal performance for BOD and TSS and their inability to effectively remove NH_3 and H_2S.

- Aquaculture and slow rate land application are technically feasible for use in Ontario but are unlikely to be economically competitive with the other alternatives.

Table 3.19
Applicability of other Lagoon Upgrading Technologies to Ontario[1]

	Intermittent Sand Filters	FWS Constructed Wetlands	SF Constructed Wetlands	Rock Filters	Aquaculture	Slow Rate Land Treatment
Number of Operating Examples	>30	>80	>80	~20	Demonstration Scale	N/A
Effectiveness for TSS Removal	Good	Good	Good	Poor	Good	Good
Effectiveness for NH_3-N Removal	Good except in cold weather	Poor unless designed for reaeration	Poor unless designed for reaeration	Poor	Marginal to excellent	Good
Effectiveness for H_2S Removal	Good except in cold weather	Poor in winter	Poor in winter	Poor	N/A	Good
Cold Weather Effects	Reduced NH_3-N and H_2S removal. Operating problems under freezing conditions	Reduced NH_3-N and H_2S removal	Reduced NH_3-N and H_2S removal	N/A	Enclosed in greenhouse	Cannot operate in subfreezing conditions
Approximate Land Rquirement	<1 ha per 1000 m^3/d capacity	6-8 ha per 1000 m^3/d capacity	3 ha per 1000 m^3/d capacity	N/A	N/A	6 - 21 ha per 1000 m^3/d capacity
Capital Costs[2] for 1000 m^3/d capacity	$152,000	$ 84,000	$ 66,500	N/A	Five to ten times other alternatives	$460,000 to $565,000
Operating Costs[2] for 1000 m^3/d capacity	$ 24,000	$ 3,700	$ 3,700	N/A	N/A	$ 22,700 to $29,000
Potential Applicability in Ontario	Yes, but on a seasonal operation basis.	Yes, but on a seasonal basis. Design upgrade required for nitrification	Yes, but on a seasonal basis. Design upgrade required for nitrification	No	Technically feasible, but more expensive than other alternatives	Yes, but on a seasonal basis and more expensive than other alternatives

Notes: 1 Reader should refer to Appendix 3 for detailed report describing these technologies.
2 Cost are in US dollars (November 1990). Q is the wastewater flow in MUSGPD. These costs are for comparative purposes only. Assumptions and applicable size ranges included in these cost estimate are included in Appendix 3.

- The intermittent sand filtration process is technically feasible for use in Ontario as demonstrated by the operation of the New Hamburg WPCP. Ammonia removal during the coldest months based on U.S. experience may be marginal. Seasonal operation may be necessary to achieve high water quality standards.

- Both types of wetlands (SWF and SF), as presently configured, provide insufficient oxygen to support the desired levels of nitrification. This is generally consistent with findings at experimental wetlands evaluated in Ontario at Listowel and Port Perry. A number of corrective measures are possible to improve performance, but seasonal operation might be required in most of Ontario to sustain high water quality standards. With these corrective measures, wetland treatment concepts are less costly to build and operate than intermittent sand filters.

- Cost analyses presented exclude the cost of land. If land costs are high, intermittent sand filtration is probably the preferred technology since it requires less land than the wetland concepts. If land costs are low, the wetland concepts may be preferred since the O&M requirements are less than those of an intermittent sand filter.

4.0 DESIGN AND COST OF "SUTTON" AND "NEW HAMBURG" PROCESS PLANTS

4.1 General Basis of Design

In order to establish capital and operating costs for generic new and retrofit "Sutton" and "New Hamburg" process plants in Ontario, cost estimates were prepared for a representative range of plant sizes. The major assumptions made in establishing the process designs are summarized herein.

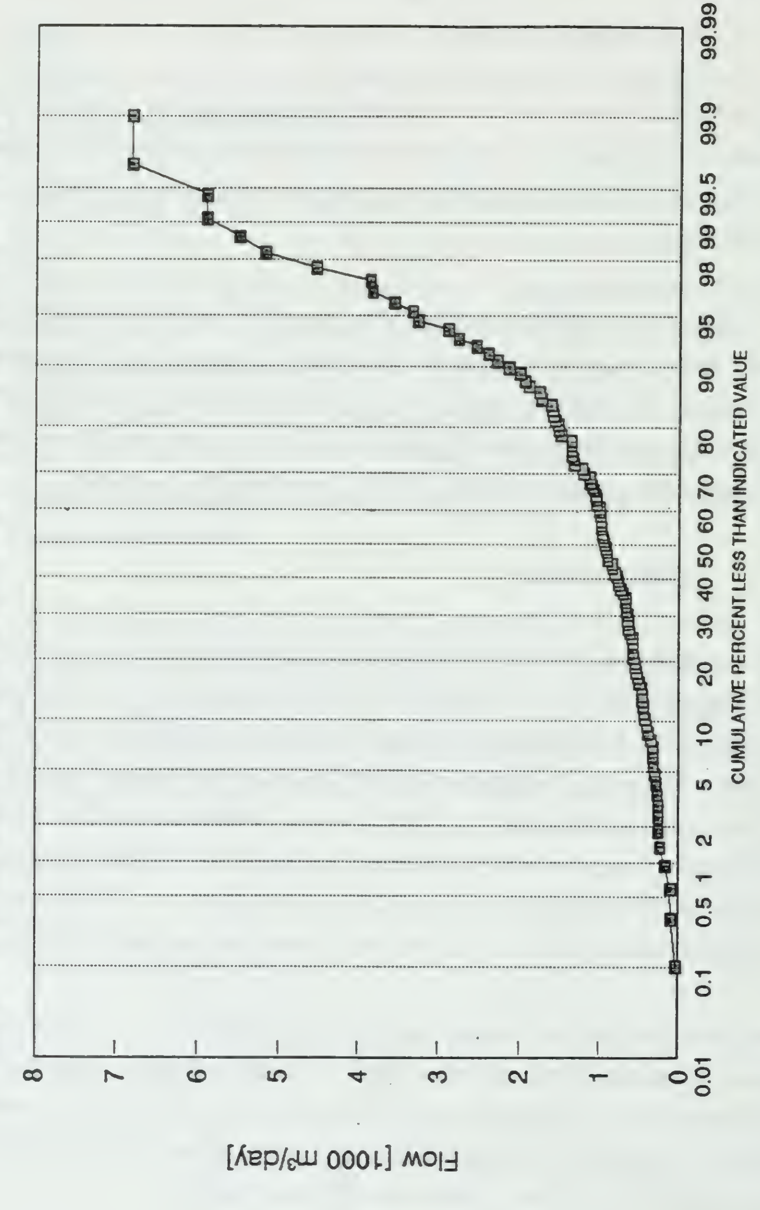
It should be emphasized that in the design of any specific Sutton or New Hamburg Process plant, the process design must reflect the site specific conditions of dry and wet weather flow, sewage strength, existing facility, etc. It is not intended that the process design developed as the basis for the generic plants costed in this report should be considered to be applicable at any particular site that is being considered for upgrading.

4.1.1 Design Capacities

In order to establish a representative size range for application of the Sutton and New Hamburg Processes in Ontario, the distribution of design capacities of existing conventional lagoons was reviewed as these would be the likely candidates for retrofit of these technologies. Figure 4.1 presents the size distribution for 109 Ontario lagoons for which dependable capacity information was available in the UMIS database. The skewed nature of the distribution is apparent with a few lagoons having a relatively large capacity compared to the majority. For costing purposes, the 10-percentile (300 m^3/d), 50-percentile (1,000 m^3/d) and 90-percentile (3,300 m^3/d) sizes were selected as the basis for design.

Maximum day flow data for existing lagoons were not available in the UMIS database. Therefore, the peaking factor for design of unit processes such as secondary clarifiers was based on the average peaking factor currently experienced at the existing Sutton Process plants where reliable flow data were available. Based on these data, the peaking factor (ratio of maximum day to average day) was established as 2.5.

Figure 4.1
ONTARIO LAGOON CAPACITY DISTRIBUTION



4.1.2 Raw Sewage Quality

For the purposes of process design, the raw sewage quality for generic new or retrofit Sutton or New Hamburg Process plants was assumed to be equal to the average raw sewage quality which has been received at conventional lagoons in Ontario over the period from 1986 through 1990. Based on data in UMIS, the following raw sewage quality was established:

BOD_5	=	135 mg/L
TSS	=	160 mg/L
Total Phosphorus	=	6.7 mg/L
Soluble Phosphorus	=	3.4 mg/L
NH_3-N	=	20.0 mg/L
TKN	=	35.0 mg/L

4.1.3 Final Effluent Quality

The effluent quality to be achieved by the conceptual designs was established by statistical analysis of the performance of current examples of each technology in Ontario. The data analysis procedures described in the Final Report of the MISA Issue Resolution Committee on Monitoring Data Analysis (MOE, 1991) were used to define the monthly average effluent quality achievable by these technologies.

4.1.3.1 Sutton Process Plants

Final effluent quality data for the Sutton WPCP, Colborne WPCP and Tottenham WPCP were used to establish the monthly performance capabilities of the Sutton Process plants. Performance data from the Cookstown WPCP were excluded from this analysis as little data were available to characterize the performance of this facility due to the seasonal nature of the discharge. In reviewing the performance data for the Sutton WPCP, it was apparent that recent effluent quality data (post-1988) reflect elevated NH_3-N concentrations in the

lagoon effluent due to ammonia release from the sludge pile as well as the effects of batch treating the lagoon with alum. To reflect the capability of the Sutton Process operated in a mode to eliminate the adverse effects of sludge accumulation in the lagoon, performance data from the Sutton WPCP from 1981 through 1988 were used along with all performance data for the Colborne WPCP and Tottenham WPCP.

Using the statistical procedures referenced above, the following monthly average effluent quality limits were established for the generic Sutton Process plants:

BOD_5	=	15.0	mg/L
TSS	=	15.0	mg/L
TP	=	1.0	mg/L
NH_3-N	=	4.0	mg/L

4.1.3.2 New Hamburg Process Plants

The effluent quality limits for the generic New Hamburg Process plants were established based on the historical performance of the New Hamburg WPCP, the only facility of this type currently operating in Ontario. Using the statistical procedures referenced above, the following monthly average effluent quality limits were established for the generic New Hamburg Process plants:

BOD_5	=	5.0	mg/L
TSS	=	10.0	mg/L
TP	=	1.0	mg/L
NH_3-N	=	4.0	mg/L

4.2 Conceptual Design and Cost of Sutton Process Plants

4.2.1 Conceptual Design

The key design criteria for the extended aeration component of each of the generic Sutton Process plants are summarized in Table 4.1. A conceptual process design flowsheet is presented in Figure 4.2.

The aeration basin design was based on the kinetic design approach suggested in the U.S. EPA Process Design Manual for Nitrogen Control (U.S.EPA, 1975). A minimum temperature of 5°C was assumed to establish the nitrification rate. A safety factor of 2.5 was applied to the minimum SRT to ensure relatively complete nitrification (0.5 to 2.5 mg/L effluent NH_3-N) at steady-state. EPA suggest a minimum safety factor of 1.5 for complete mix activated sludge processes under these conditions (U.S.EPA, 1975). Based on this design procedure, the design conditions presented in Table 4.1 were established for the aeration basin. These design criteria are compared to other nitrification system aeration designs and to the actual design of existing Sutton Process plants in Table 4.2. The design is conservative compared to MOE Guidelines in terms of SRT and organic loading but has a shorter HRT at 12 hours than any of the existing examples of Sutton Process plants.

Average oxygen demand for the generic Sutton Process extended aeration plants was based on the Boon equation (Boon and Chambers, 1982). A peak hourly oxygen demand of 1.7 times the average was applied. No oxygen credit was assumed for simultaneous denitrification in the process.

Clarifier design was based on a peak day hydraulic loading (peaking factor of 2.5) of 35 m^3/m^2.d and a solids loading at peak flow and 100 percent recycle of 160 kg/m^2.d. These design criteria are compared to other recommended design standards and to the designs of existing Sutton Process secondary clarifiers in Table 4.3.

TABLE 4.1

SUMMARY OF SUTTON PROCESS DESIGNS

		@ 300 m^3/d	@ 1,000 m^3/d	@ 3,300 m^3/d
1.	Average Flow (m^3/d)	300.	1,000.	3,300.
2.	Peak Day Flow (m^3/d)	750.	2,500.	8,250.
3.	BOD_5 Loading (kg/d)	40.5	135.	446.
	TSS Loading (kg/d)	48.0	160.	528.
	TKN Loading (kg/d)	10.5	35.	116.
4.	Aeration Section			
	Basin Vol (m^3)	150.	500.	1,650.
	Avg. O_2 Required (kg/h)	4.1	13.7	45.1
	Peak O_2 Required (kg/h)	7.0	23.3	76.7
	MLSS (mg/L)	3,300.	3,300.	3,300.
5.	Phosphorus Removal			
	Alum Feed (ml/hr)	1,835.	6,120.	20,200.
6.	Clarification			
	Surface Area (m^2)	21.5	71.4	235.7
	Recycle Rate (m^3/d)	150 - 600.	500 - 2,000.	1,650 - 6,600.
	SLR ($kg/m^2.d$)	160.	160.	160.
7.	Estimated Sludge Production kg/d TS	20.	60.	200.

Figure 4.2 PROCESS FLOWSHEET – SUTTON PROCESS FACILITIES

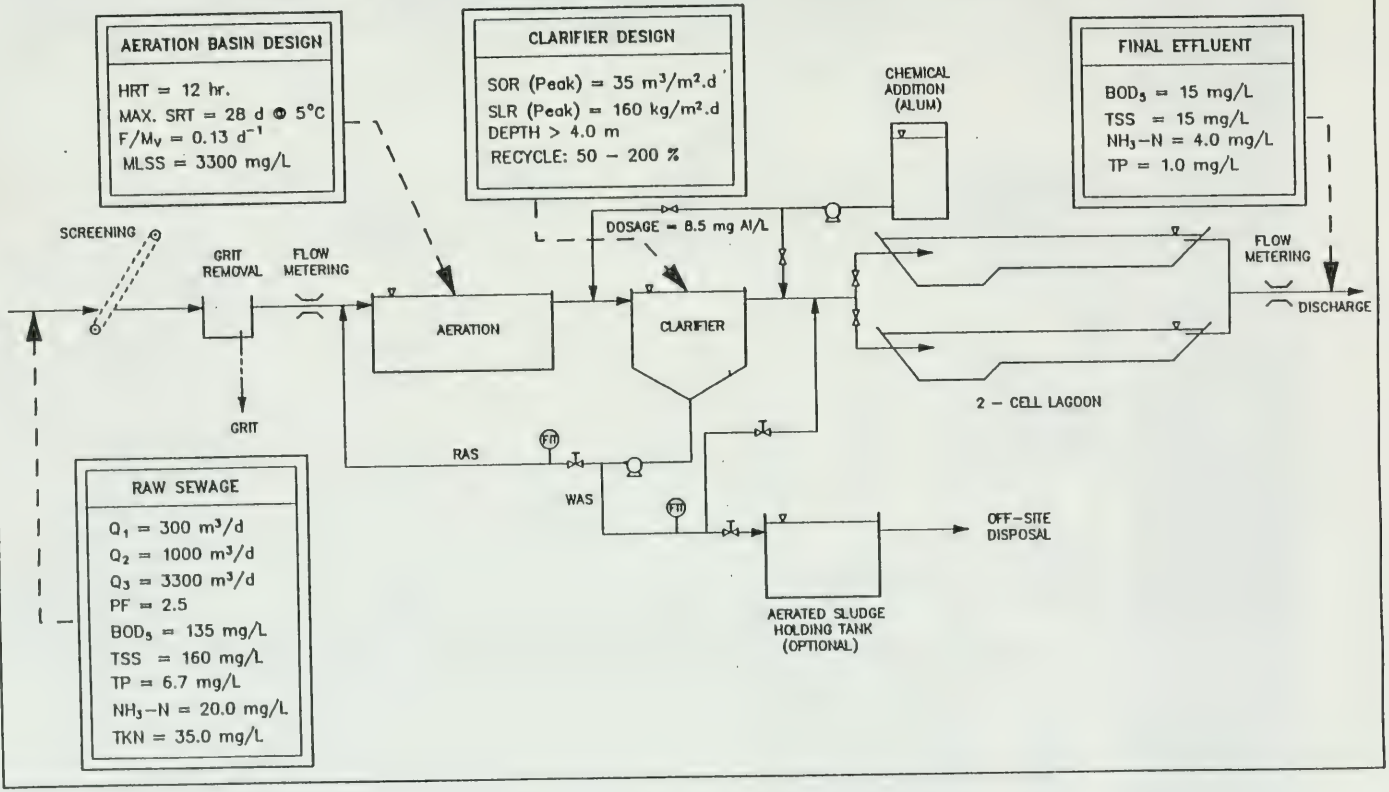


TABLE 4.2

PROCESS DESIGN CRITERIA FOR
SUTTON PROCES AERATION BASIS

Parameter	Design Basis			
	This Report	MOE Guidelines (MOE 1984)	Sutton, Jank, Monaghan and Murphy (1979)	Existing Sutton Process Plants
HRT (h)	12.	> 15	6.8	16-55
Max. SRT (d)	28.	> 15	6.8	> 50
F/Mv (d^{-1})	0.13	0.05 - 0.15	0.24	< 0.03
MLSS (mg/L)	3300	--	--	4000 - 7000

TABLE 4.3

PROCESS DESIGN CRITERIA FOR SUTTON PROCESS SECONDARY CLARIFIERS

Parameter	Design Basis						
	This Report	MOE Guidelines	U.S. EPA Suspended Solids Removal Manual	Ten State Standards	MOP No. 8	U.S. Army	Existing Sutton Process Plants
Surface Loading Rate ($m^3/m^2.d$)							
Peak Day	35.0	< 35	33	41	--	33	24-35
Average Day	14.0	--	8-16	--	33	24	5-17
Solids Loading Rate ($kg/m^2.d$)							
Peak Day	160	< 120	245	245	--	--	--
Average Day	--	--	98 - 147		49 - 290	--	--
Depth (m)	> 4.0	3.6 - 4.6	3.7 - 4.6	> 3.7	3 - 4.6	2.4 - 4.3	--

Phosphorus removal was based on alum addition to the aeration basin at a dosage equal to an aluminum to soluble phosphorus (Al:SP) weight ratio of 2.5 based on the raw sewage SP concentration to provide 90 percent removal of soluble phosphorus (U.S.EPA, 1976).

4.2.2 Design and Costing Assumptions

The design criteria used for the Sutton Process plants has been assembled based upon results of the process review, individual site visits, and consultations with various consultants and plant operators. The resultant criteria is an attempt to establish a "generic" design upon which costs can be based. In order to apply the costing and design information contained herein to a specific design, the reader should be cognizant of the design assumptions listed here.

The Sutton Process has been indicated previously on the process flow diagram in Figure 4.2. Specific design assumptions have been listed in Appendix 4. Preliminary treatment would consist of manual bar screens and grit channels. The aeration cell would be either an earthen basin or concrete tanks, and be complete with a bridge mounted mechanical surface aerator. Two circular concrete clarifiers would be provided.

Return sludge capabilities and a chemical feed system would be housed in a central control building. The building would also house local control facilities, as well as a laboratory, washroom and office for the full time operator.

For retrofit facilities, the assumption has been made that a lagoon with at least 60 day storage exists on site. This lagoon would be bermed off to create two identical lagoons. Depending upon which method of sludge disposal was incorporated into the design, sludge would be discharged directly to the lagoon (as in the "traditional" Sutton concept), or be discharged to a separate sludge holding tank. For discharge directly to the lagoon, a local "pod" would be constructed in the vicinity of the sludge discharge, sized to store a 5 year accumulation of sludge at a 10% sludge concentration. This is to allow for removal of sludge from the lagoons every 5 years (it was previously noted in Section 3.4.1 that ammonia

is solubilized from the sludge, increasing to significant levels after 6-7 years of operation). The feasibility of sludge removal from lagoons is further discussed in Section 5, Additional Information Needs. For discharge to a sludge holding tank, the holding tank would be sized for 35 days sludge age at a 4% sludge concentration. The tank would be complete with a mixer to provide mixing aeration, and would have supernating capabilities. An additional portion of the existing on-site lagoon would be bermed off to provide 180 days on-site sludge storage. Ultimate sludge disposal would be by means of land spreading.

For new facilities, lagoons would be sized to provide 60 days (continuous discharge) storage. Special artificial liners would not be provided, since is assumed that the soils condition would be similar to compacted clay.

4.2.3 Costs of Sutton Process Plants

Costs for facilities using the above design assumptions have been compiled based upon current industry standards. All costs are in 1992 dollars, using a March, 1992 ENR index = 6537.

Capital costs of Sutton Process Plants have been indicated in Figure 4.3, which include costs for both new facilities and retrofit facilities, with and without sludge discharge to the polishing lagoons. As indicated by the Figure, capital costs for new facilities range from \$1,400,000 to \$3,900,000 for the 300 m^3/d to 3300 m^3/d facilities respectively, while the retrofit facilities range from \$1,100,000 to \$2,500,000. This is regardless of either sludge handling method, since this has little impact on total capital cost. A more detailed capital cost breakdown has been included in Appendix 4, Table A4.8. As was mentioned in Section 4.2.2, the aeration cells for these plants could be either concrete tanks or earthen basins. For the costs presented herein, however, costs of aeration systems have been based upon concrete tanks. For use with earthen basins, total capital costs would

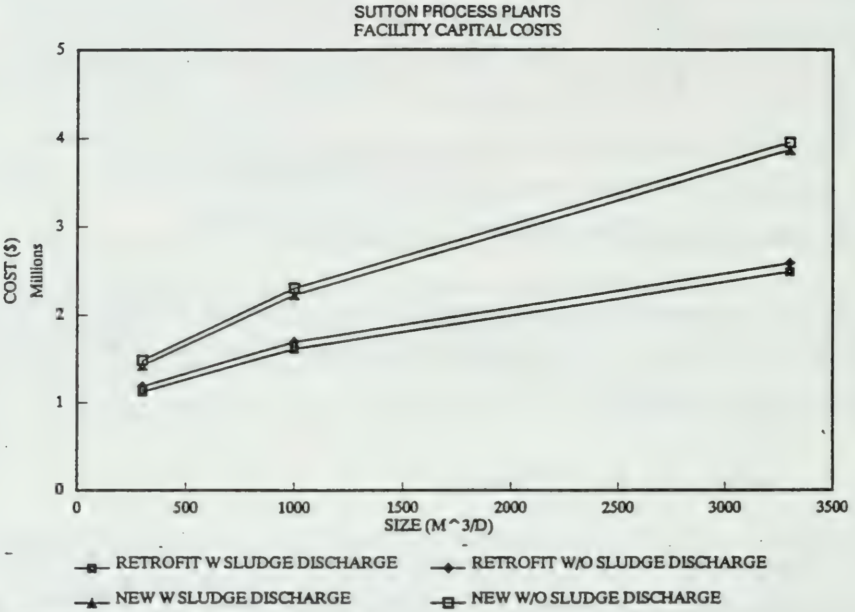


FIGURE 4.3

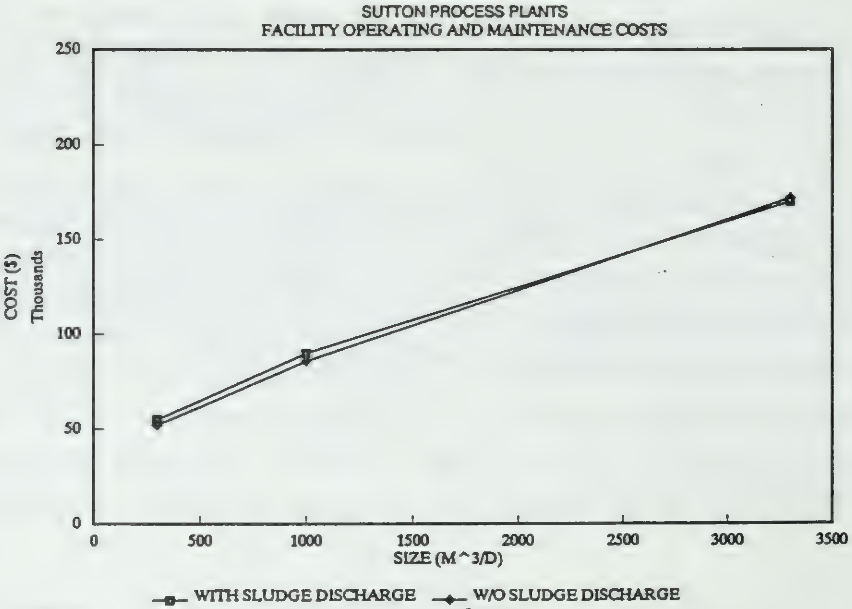


FIGURE 4.4

decrease by \$46,000, \$97,000 and \$210,000, respectively for the 300 m^3/d, 1000 m^3/d and 3300 m^3/d sized plants.

Figure 4.4 indicates the approximate yearly operating and maintenance costs for the various sized facilities. These generally range from \$55,000 to \$170,000 per year respectively for the 300 m^3/d to 3300 m^3 sized facility. The operations cost for sludge discharge to lagoons includes all annual costs such as hydro, chemicals, labour etc., plus the cost of removing sludge every 5 years (the 5 year sludge removal cost has been converted to a yearly cost). As can be seen by the figure, there is virtually no difference in operations and maintenance costs between the two methods of sludge handling.

Table 4.4 indicates the different annual capital and operation and maintenance costs for the facilities. The retrofit facilities are 15% - 25% less expensive (on an annual basis) than the new facilities.

TABLE 4.4

SUTTON PROCESS PLANTS
ANNUAL CAPITAL AND OPERATIONS AND MAINTENANCE COSTS*
($/m^3/d$)

	300 m^3/d	1000 m^3/d	3300 m^3/d
New Plants - Sludge to Lagoon	667	317	171
New Plants - Sludge to Holding Tank	678	321	174
Retrofit Plants - Sludge to Lagoon	565	254	128
Retrofit Plants - Sludge to Holding Tank	577	258	132

* 1992 dollars, ENR Index = 6537

4.3 Conceptual Design and Cost of New Hamburg Process Plants

4.3.1 Conceptual Design

The conceptual design of the generic New Hamburg Process plants were based on the design of the New Hamburg WPCP. A conceptual design flowsheet is presented in Figure 4.5.

Filter area was based on a nominal area of 1.8 m^2 per m^3/d of design capacity. It was assumed that the filter would operate from May to October for a total of 1200 hours, consistent with the current operation at New Hamburg. Winter storage would be provided in the facultative lagoons.

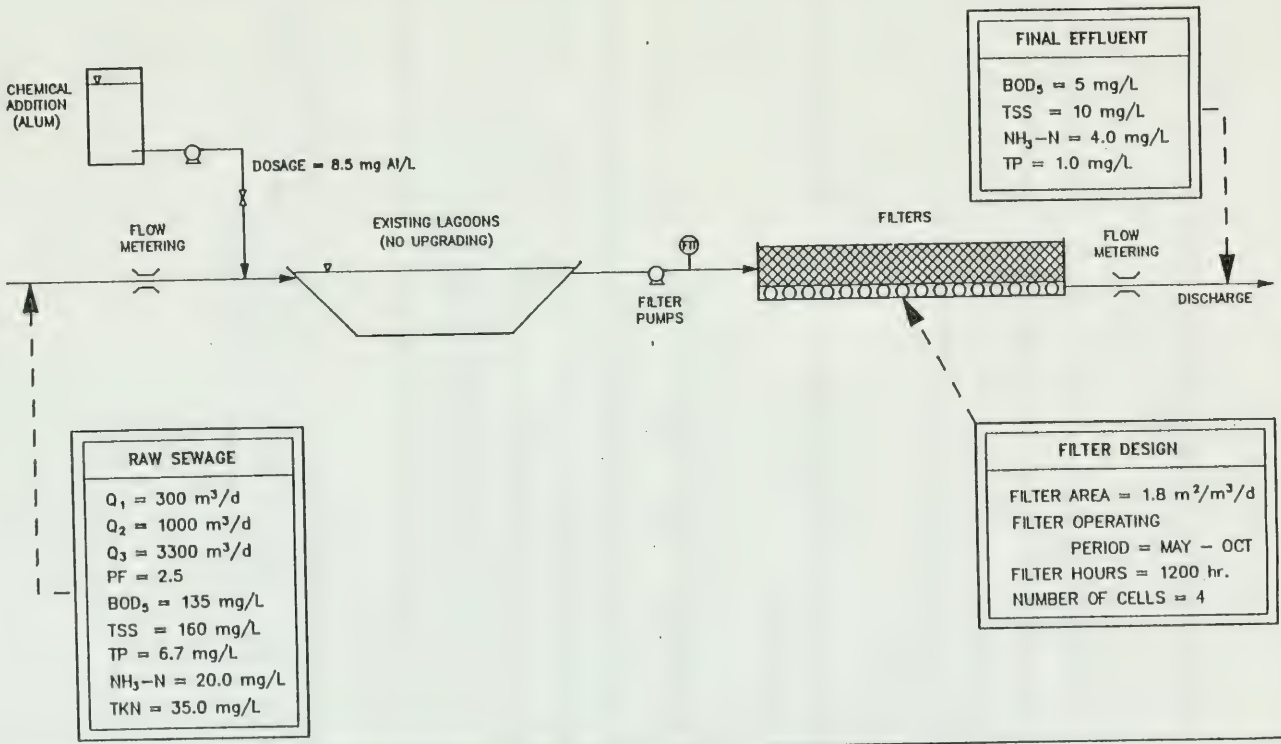
Phosphorus removal would be provided by alum addition to the lagoon influent. The chemical dosage was based on an aluminum to soluble phosphorus weight ratio of 2.5.

4.3.2 Design and Costing Assumptions

The New Hamburg Process plant design has been defined above and shown in Figure 4.5. Preliminary treatment is not required, since the first unit process is facultative lagoons. Aerated lagoons, which were installed in the plant in New Hamburg, but not in Schomberg, have not been included (the aerated lagoons at the New Hamburg WPCP were to treat the relatively high BOD_5 loading to the plant). The filters are sized based upon the loading rates experienced in New Hamburg, using a layered sand and gravel media. The spray application to the filters is via perforated steel pipe laid across the tops of the filter, while the filter underdrain system is a series of perforated PVC pipes at the bottom of the media. The flow into the plant is via gravity, with pumps utilized to pump both from the lagoons to the filter media and from the filter underdrain system into the receiving waterbody.

Chemical addition for phosphorus removal would be at the inlet to the lagoons. Chemical storage would be adjacent to a control building. This control building will house the

Figure 4.5 PROCESS FLOWSHEET – NEW HAMBURG PROCESS FACILITIES



chemical feed pumps, control equipment, as well as a small laboratory and office.

For the retrofit plants, the assumption is made that a lagoon exists on-site with an equivalent 180 day seasonal discharge storage. This is to accommodate the additional storage needed since spray application to the filter beds can only occur 6 to 8 months of the year. For new plants, lagoons would be designed for 180 days storage using MOE design guidelines for stabilization ponds. Again, as in the case of Sutton, no special artificial liner would be provided.

No costs are included for replacement of filter media or for removal and disposal of sludge from the lagoon system.

4.3.3 Costs of New Hamburg Process Plants

Figure 4.6 indicates the capital costs of both new and retrofit facilities for various sizes. The costs of new facilities range between \$1,000,000 for the smaller sizes to \$4,300,000 for the larger, while the retrofit facilities range between \$500,000 to \$1,000,000. The rather steep slope on the new facilities curve as compared to the retrofit curve is primarily a function of the additional cost incurred by providing seasonal discharge lagoons for the new facilities. Detailed capital costs breakdowns have been indicated in Appendix A, Table A4.9.

Annual operating and maintenance costs for the New Hamburg process plants are shown in Figure 4.7. These range from \$35,000/yr to \$90,000/yr for the range of plants. These cost should not differ between new plants and retrofit plants.

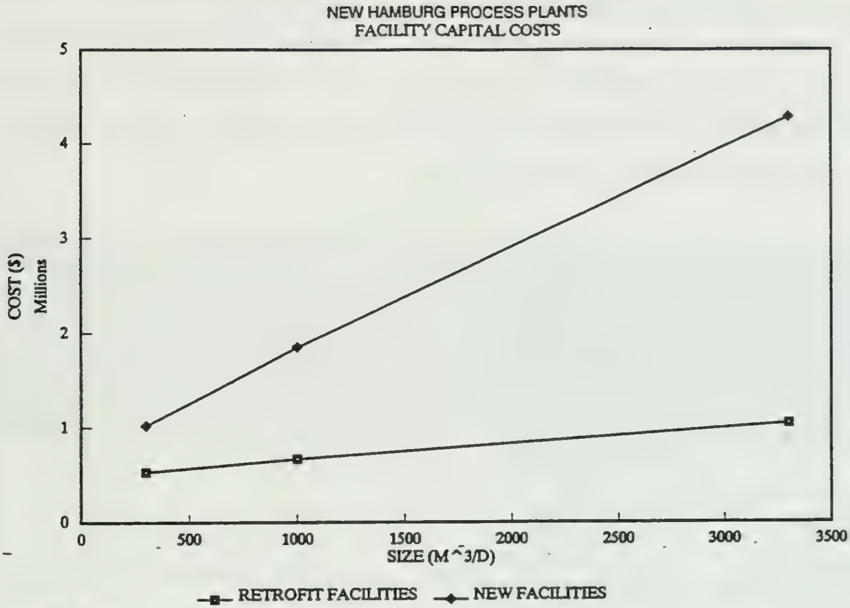


FIGURE 4.6

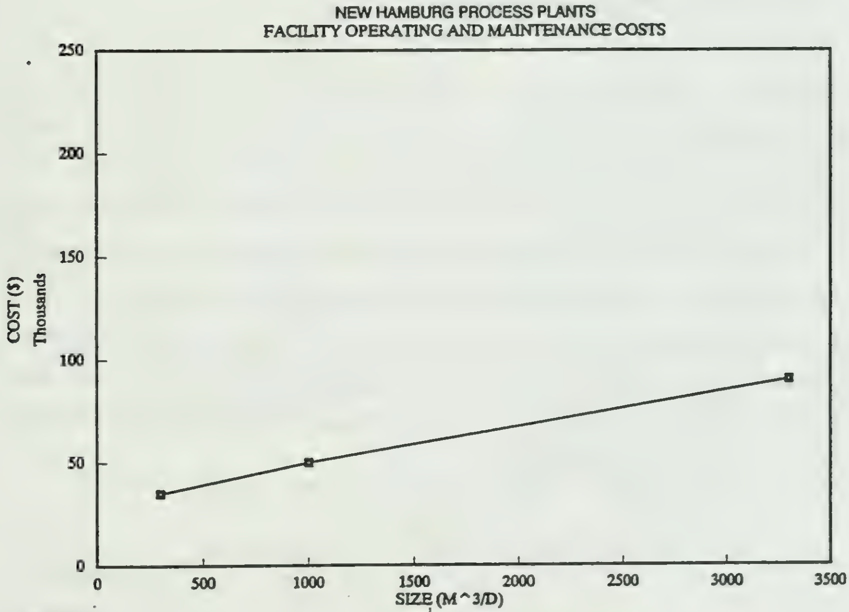


FIGURE 4.7

Yearly amortized capital and operations and maintenance costs for the New Hamburg Process facilities have been indicated in Table 4.5. The annual costs for new plants range from \$463/m^3.d (\$2,100,000/MGD) for the 300 m^3/d sized plant to \$160/m^3.d (\$730,000/MGD) for the 3300 m^3/d plant, while the retrofit plants range from \$296/m^3.d (\$1,300,000/MGD) to \$59/m^3.d (\$270,000/MGD).

TABLE 4.5
NEW HAMBURG PROCESS PLANTS
ANNUAL CAPITAL AND OPERATIONS AND MAINTENANCE COSTS*
(\$/m^3.d)

	300 m^3/d	1000 m^3/d	3300 m^3/d
New Plants	463	238	160
Retrofit Plants	296	118	59

* 1992 dollars, ENR index = 6537

4.4 Cost Comparison and Discussion

Lifecycle costs were calculated in order to compare capital and operating costs for the various lagoon systems presented previously. Operating and maintenance costs were added to the amortized capital cost, using a 20 year design life and an interest rate of 8%. These lifecycle costs for both Sutton Process Plants and New Hamburg Process Plants have been compared with that of conventional lagoons (seasonal and continuous discharge) for both new and retrofit facilities, and have been indicated by the cost curves in Figures 4.8 and 4.9. The lowest cost alternative appears to be either conventional lagoons (continuous discharge) or retrofit New Hamburg Process Plants, while the highest cost alternative is New Sutton Process Plants. When comparing retrofit facilities versus new facilities, the New Hamburg Process plants are lower in cost for all sizes.

NEW FACILITIES (CONVENTIONAL LAGOONS, SUTTON AND NEW HAMBURG PLANTS)
AMORTIZED CAPITAL AND O & M COSTS

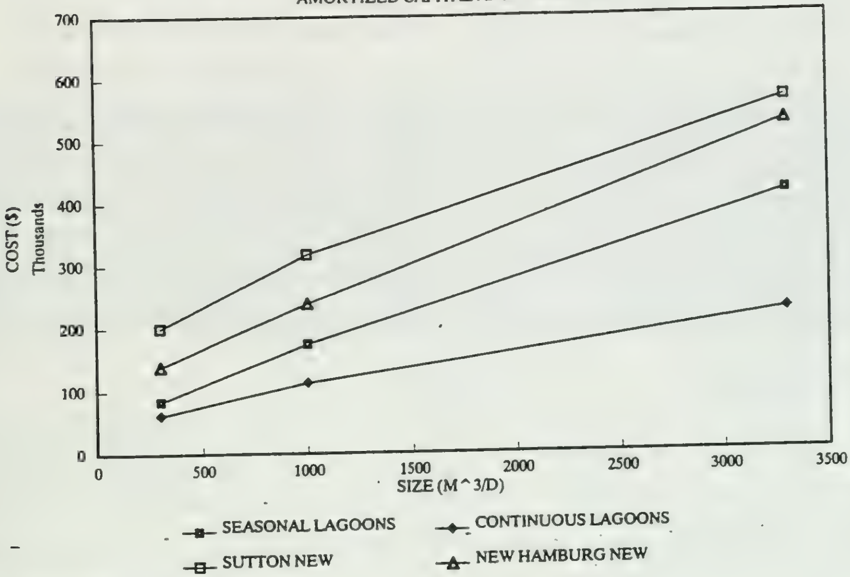


FIGURE 4.8

RETROFIT FACILITIES (SUTTON AND NEW HAMBURG PLANTS)
AMORTIZED CAPITAL AND O & M COSTS

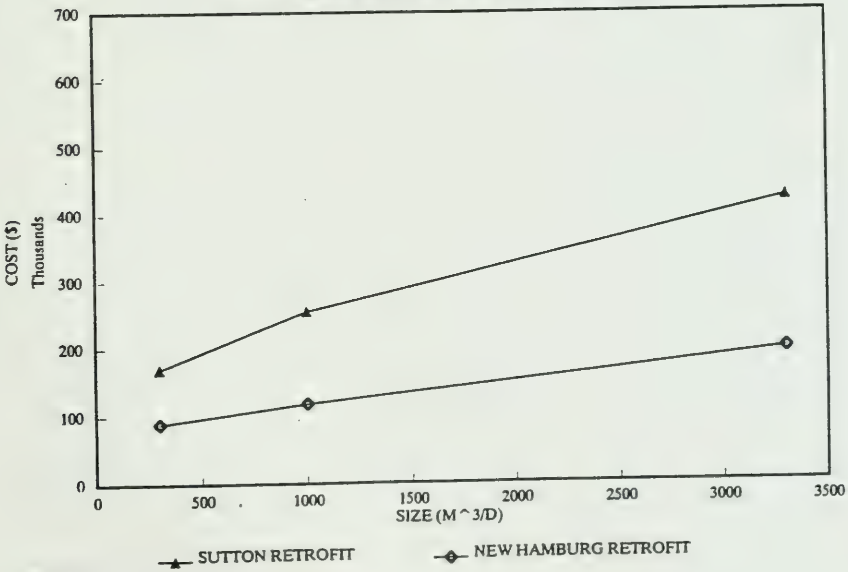


FIGURE 4.9

Using the results of the cost estimate presented herein, approximate capital costs of upgrading conventional lagoons across Ontario to either Sutton or New Hamburg process facilities have been calculated and presented in Table 4.6. These costs are based upon unit costs for the various facilities (in $\$/m^3.d$) and the total design capacity for lagoon systems in Ontario (MOE, 1991) using the approximate size distribution of lagoons as presented in Section 4.1.1 (i.e 10% of existing lagoons are in the 300 m^3/d range, 80% are in the 1000 m^3/d range, and 10% are in the 3300 m^3/d range). It should be noted that only existing conventional lagoons (continuous, seasonal or annual discharge) in Ontario were included in this table (lagoons with aerated cells or those without discharge to surface waters were excluded).

TABLE 4.6 APPROXIMATE CAPITAL COSTS OF UPGRADING CONVENTIONAL LAGOONS IN ONTARIO TO SUTTON AND NEW HAMBURG PROCESS FACILITIES* ($1,000,000's)		
	Sutton Process Facilities	New Hamburg Process Facilities
Northern Ontario†	134	57
Southern Ontario†	130	55
TOTAL	264	112

* 1992 dollars, ENR index = 6537

† for existing conventional lagoons (continuous, seasonal and annual discharge) only

5.0 ADDITIONAL INFORMATION NEEDS

The previous analyses and costing is based upon available information obtained during this study. In order to fill any gaps in information, assumptions were made. Additional information is required to increase the accuracy of costs and to address other issues which were raised but not resolved. The following sections identify these information needs both with respect to the Sutton Process and the New Hamburg Process.

5.1 Sutton Process

- Sludge disposal into the stabilization ponds results in increased ammonia levels and ultimately increased NH_3-N in the effluent, due to resolubilization of ammonia from the sludge pile. This would necessitate the removal of sludge in 5-6 years, depending upon the size of the lagoon. Currently, there is not enough information available to determine if re-distributing the sludge so that it does not form an anaerobic pile will result in a decreased rate of ammonia solubilization. This issue needs to be investigated to determine if the sludge removal time could be extended in this operating mode. If this is effective, a separate and more effective sludge/liquid distribution system would need to be designed and the cost implications of this distribution system considered within the overall Sutton Process costs.

- Further to the above, there is a need to demonstrate the feasibility of sludge removal from an existing Sutton Process lagoon to determine the most effective method and the resultant cost. This will allow a more accurate cost/benefit analysis to define the economic advantages of the Sutton Process. Additionally, the suggested sludge removal frequency of once every five years has been selected based upon performance data at the Sutton WPCP. To confirm this frequency, additional information regarding the rate of ammonia release and factors affecting the rate of release are needed. This will establish the feasibility of operating in the Sutton mode.

- Elevated total phosphorus concentrations were observed at the Sutton WPCP. However, facilities such as the Colborne WPCP, which have been operating for almost as long as the Sutton WPCP, do not exhibit such trends. More detailed investigations need to be conducted to determine the cause of this phenomenon at the Sutton WPCP.

- The data available on which to base an evaluation of the toxicity of Sutton Process plant effluents are limited. These data are all based upon acute toxicity tests which are relatively non-responsive. To assess the benefits of the Sutton Process in terms of toxicity reduction, additional data are needed. This should include sub-lethal testing as well as acute toxicity testing, spanning all seasonal periods (cold and warm weather).

- The design of the individual unit processes within the Sutton process plants are conservative in terms of hydraulic loading. There is insufficient information to define the minimum aeration tank HRT, and SRT, to maintain cold weather nitrification, maximum clarifier loading or minimum lagoon size needed to achieve the desired effluent quality. Stress testing should be conducted at an existing plant to determine the required design criteria and to optimize process operating conditions, thus maximizing plant performance.

- There are currently no data available to define the minimum TP concentration achievable from a Sutton Process Plant.

- There are currently no examples of the Sutton Process operating under the extreme weather conditions experienced in Northern Ontario. Therefore, the effects of extended ice cover of the lagoon on ammonia release from the sludge and H_2S production cannot be predicted from the available information.

- It was not possible to compare the performance of Sutton Process plants with extended aeration plants followed by polishing ponds which do not receive sludge because there are no examples of this type of plant in Ontario. Therefore, the benefits of discharging sludge to the lagoons could not be defined.

5.2 New Hamburg Process

- The only experience with the New Hamburg Process is based upon the design and operation of the New Hamburg WPCP. Monitoring of the Schomberg WPCP should be considered in order to provide additional performance, operational and cost data.

- The design basis for the filters at the generic "New Hamburg" process plants was based upon the design of the New Hamburg WPCP, which is consistent with designs of similar plants in the U.S. However, the minimum filter area, maximum filter loads, and filter media specification for New Hamburg process plants in Ontario have yet to be established. Stress testing at an existing facility would be required to establish the optimum design. Furthermore, possible effects on filter operation and performance of alternate filter liquid application designs need to be evaluated, with particular emphasis on nitrification efficiency.

- Additional comparative testing of the acute and chronic toxicity of the lagoon effluent and filter effluent should be conducted at the New Hamburg WPCP under different seasonal conditions to establish the extent of toxicity reduction achieved by the New Hamburg Process.

- There are currently no data available to define the minimum TP concentration achievable by the New Hamburg process. Optimization of phosphorus removal in this process should include an evaluation of the

effectiveness of chemical addition to the filter influent and must consider the long term effects of this operating mode on filter operation.

- There are currently no examples of the New Hamburg Process operating under the extreme climatic conditions experienced in Northern Ontario. Therefore, it is not possible to evaluate the effects of extended winter shutdown of the filter on the time required to re-initiate effective treatment, particularly with respect to nitrification.

- Limited data exist to confirm the time period required to initiate and sustain nitrification within the filter media at start-up in the spring after winter shutdown. A well-designed monitoring program should be conducted at the New Hamburg WPCP to document filter performance at start-up under cold weather conditions.

5.3 Other Upgrading Alternatives

- Although alternatives other than the "Sutton" or "New Hamburg" concept have been reviewed in this report, further investigation is required to assess the technical feasibility and economics of aquaculture and land treatment systems for application in Ontario.

- Further to the above, the practical and economic approaches for upgrading wetland concepts to provide the desired level of nitrification under Ontario operating conditions needs to be developed further and demonstrated.

6.0 CONCLUSIONS

6.1 Performance Capabilities

- Based on existing examples of conventional lagoons in Ontario, these systems are capable of achieving the following effluent limits on a monthly average basis:

Effluent Quality (mg/L)

Process	BOD_5	TSS	TP	NH_3-N
Continuous Discharge	30	40	N/A	10.0
Annual Fill-and-Draw	15	65-90	1.5	4.0
Seasonal Fill-and-Draw	15-30	25-50	1.0-1.5	6.0-14.0
Aerobic Facultative	20-25	30-50	1.5	9.0-16.0

The systems are subject to significant seasonal effects, including elevated ammonia concentrations in spring discharges. Elevated suspended solids concentrations in summer and fall discharges due to algal growth and in spring discharges due to the lagoon turnover were also noted.

- The Sutton and New Hamburg Processes are effective in upgrading the quality of effluents from conventional lagoon systems to better than secondary treatment levels. The performance capabilities of these systems on a monthly average basis are as follows, based on available performance data for three Sutton Process plants (Sutton WPCP, Colborne WPCP and Tottenham WPCP) and one New Hamburg Process plant (New Hamburg WPCP):

Effluent Quality (mg/L)

Process	BOD_5	TSS	TP	NH_3-N
Sutton Process	15	15	1.0	4.0
New Hamburg Process	5	10	1.0	4.0

- An increase in the ammonia concentration in the Sutton WPCP lagoon effluent was apparent after about five to seven years of operation. This increase was related to the release of ammonia from the sludges discharged to the lagoon. A similar ammonia release was measured at Colborne and Tottenham in the sludge pile. Increases in phosphorus concentrations were also evident in the Sutton WPCP lagoon but were not identified in any of the other Sutton Process plants. To maintain the effluent quality from these systems, it will be necessary to remove sludge from the lagoon and dispose of it on a regular basis. Current information suggests that a sludge removal frequency of five to seven years may be necessary depending on the quantity of sludge discharged to the lagoon and the relative lagoon size.

- Based on limited data, effluents from all of the operating Sutton process plants were not acutely toxic to rainbow trout and Daphnia magna. Data from the New Hamburg WPCP suggests that a reduction in effluent toxicity is accomplished across the filter in the spring. Overall, there are insufficient data on the toxicity of effluents from conventional lagoons, Sutton Process plants and New Hamburg Process plants on which to base firm conclusions regarding the relative toxicity of these effluents.

- Aquaculture and slow rate land application are other processes which would be technically feasible for use in Ontario to upgrade lagoon effluent quality. Designs of wetland systems would need to be modified to provide increased oxygen transfer in order to support the desired levels of nitrification.

6.2 Design Considerations

6.2.1 Sutton Process

- Most existing Sutton Process plants do not presently provide preliminary treatment. However, based on the operational problems of those plants which do not have either manual bar screens or grit channels, namely clogged return sludge pumps and surface aerators, preliminary treatment should be provided.

- The extended aeration component of most existing Sutton process plants have been designed very conservatively in terms of aeration basin and clarifier loading. This is especially evident with the more recent examples.

- The current configuration of lagoon designs used with the Sutton process leads to ammonia release from the settled waste sludges after 5-6 years of operation. Based upon available information from the Sutton WPCP, phosphorus resolubilization may also be a problem.

- The selection of aeration hardware requires attention in order to minimize operation and maintenance problems which exist in some of the present Sutton Process systems.

- Since the Sutton Process was developed as a "low cost" alternative to upgrade conventional lagoon systems in smaller municipalities, lower cost construction approaches, such as the use of earthen aeration basins, need to be considered in the design.

- Sludge removal from lagoons on a periodic basis, is required in order to avoid future degradation of effluent quality. Plans for sludge removal and ultimate disposal are needed prior to implementing a Sutton process system.

- In order to accommodate sludge removal on a periodic basis, lagoon systems need to be designed with at least two lagoons in parallel.

- To allow servicing of the extended aeration plant, two clarifiers should be provided.

6.2.2 New Hamburg Process

- Because of the seasonal nature of the filter operation, plants using the New Hamburg process need to be designed to accommodate winter storage.

- Design of lagoons for these systems should be consistent with current conventional lagoon design practices used in the industry. This can be tailored to include aerated cells should the wastewater characteristics require this form of pretreatment, providing land availability and cost considerations are accounted for.

- Design of effluent filters must consider both the seasonal and nighttime limitations of its operation.

6.3 Costs

- Based upon the design assumptions stated herein, the estimated capital, operation and maintenance, and lifecycle costs for the 300 m^3/d to 3300 m^3/d sized plants are as follows (using a 20 year design life, at an amortization rate of 8%, ENR index = 6537):

PLANT TYPE AND SIZE	CAPITAL COST ($)	OPERATION/ MAINTENANCE COSTS ($/YR)	LIFECYCL E COSTS ($/YR)
Sutton Process Plant (New)			
300 m^3/d	1,400,000	55,000	200,000
1000 m^3/d	2,200,000	90,000	320,000
3300 m^3/d	3,900,000	170,000	560,000
Sutton Process Plant (Retrofit) 300 m^3/d			
1000 m^3/d	1,100,000	55,000	170,000
3300 m^3/d	1,600,000	90,000	250,000
	2,500,000	170,000	420,000
New Hamburg Process Plant			
(New) 300 m^3/d	1,000,000	35,000	140,000
1000 m^3/d	1,900,000	50,000	240,000
3300 m^3/d	4,300,000	90,000	530,000
New Hamburg Process Plant			
(Retrofit) 300 m^3/d	500,000	35,000	90,000
1000 m^3/d	700,000	50,000	120,000
3300 m^3/d	1,000,000	90,000	200,000

- The New Hamburg Process Plant is the less expensive alternative when compared to the Sutton Process Plant, for both new and retrofit facilities. The costs for New Hamburg Process Plants are, however, based upon a 6 month discharge period, and may not necessarily be less expensive if a shorter discharge period is used.

- Operating and maintenance costs for the Sutton Process Plant will be higher than that for the New Hamburg Process plant because of the costs of operating the aeration system, providing a full time operator, and the yearly sludge disposal costs. The appropriate percentage of operation and maintenance costs related to sludge removal from lagoons for various sizes of Sutton Process Plants is as follows:

300 m^3/d	-	9%
1000 m^3/d	-	11%
3300 m^3/d	-	12%

6.4 Other Conclusions

- In addition to being an effective and low cost alternative for upgrading existing lagoon systems, the New Hamburg Process plants are also "aesthetically pleasing" in that they blend in well with the natural surroundings of a rural or semi-rural environment.

- Aquaculture and land application concepts are technically feasible for use in Ontario climatic conditions for upgrading conventional lagoons to reduce seasonal TSS, ammonia and H_2S problems. However, these processes cannot be operated in sub-freezing conditions and are unlikely to be economically competitive with the New Hamburg process.

- Wetland concepts as treatment systems require further development to ensure that the desired levels of nitrification are achieved.

7.0 Recommendations

- The New Hamburg Process produces a higher quality effluent at a lower annual cost than the Sutton Process. The New Hamburg Process is therefore recommended as the preferred alternative for upgrading conventional lagoon systems, although site specific conditions must be considered in the final process selection. Climatic conditions, receiving stream assimilative capacity, seasonal discharge limitations, site conditions, and other factors need to evaluated in each case.

- Should a Sutton process design be considered, it is recommended that attention be paid to sludge handling and its negative impact on effluent quality.

- Further investigation is required to address the information needs as identified in Section 5.0.

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A1.1 SUTTON WPCP

Background

The Sutton Process was implemented at the Sutton WPCP in 1981. The original plant consisted of a single cell seasonal discharge facultative lagoon. In 1976, extended aeration was added resulting in an operating concept of a conventional extended aeration plant with a polishing lagoon. The only modification to the lagoon was a change to the inlet piping. No sludge was removed from the lagoon at that time. Phosphorus removal was implemented by the addition of ferric chloride to the plant pumping station. Ferric chloride was eventually changed to alum and the addition point was changed to the effluent of the aeration tank. Waste sludge was stored in an aerated tank until it was disposed of on agricultural land. In 1981, the MOE began to experiment with the Sutton Process by wasting the excess sludge directly to the lagoon. The aerated sludge holding tank was taken out of service at that time.

Process Description

A flow schematic of the Sutton WPCP is shown in Figure A1.1. Flow is delivered to the plant by an off-site on-off pumping station. Flows are measured by a Parshall flume. Flow measurement accuracy is questionable due to the physical configuration of the flume approach and the extensive flow surges from the on-off pump station operation. There are no pre-screening or grit removal facilities.

The earthen aeration basin has a synthetic liner and is about 3.7 m deep with a total volume of approximately 2046 m^3. Oxygen is supplied by an 18.6 kW low speed surface aerator with a submerged propeller. Alum can be added to the aeration tank effluent prior to clarification or to the clarifier effluent for phosphorus removal.

The single rectangular clarifier has a surface area of 120 m^2 and a volume of 318 m^3. Clarified effluent is discharged directly to a single lagoon which has an area of 6.5 ha and

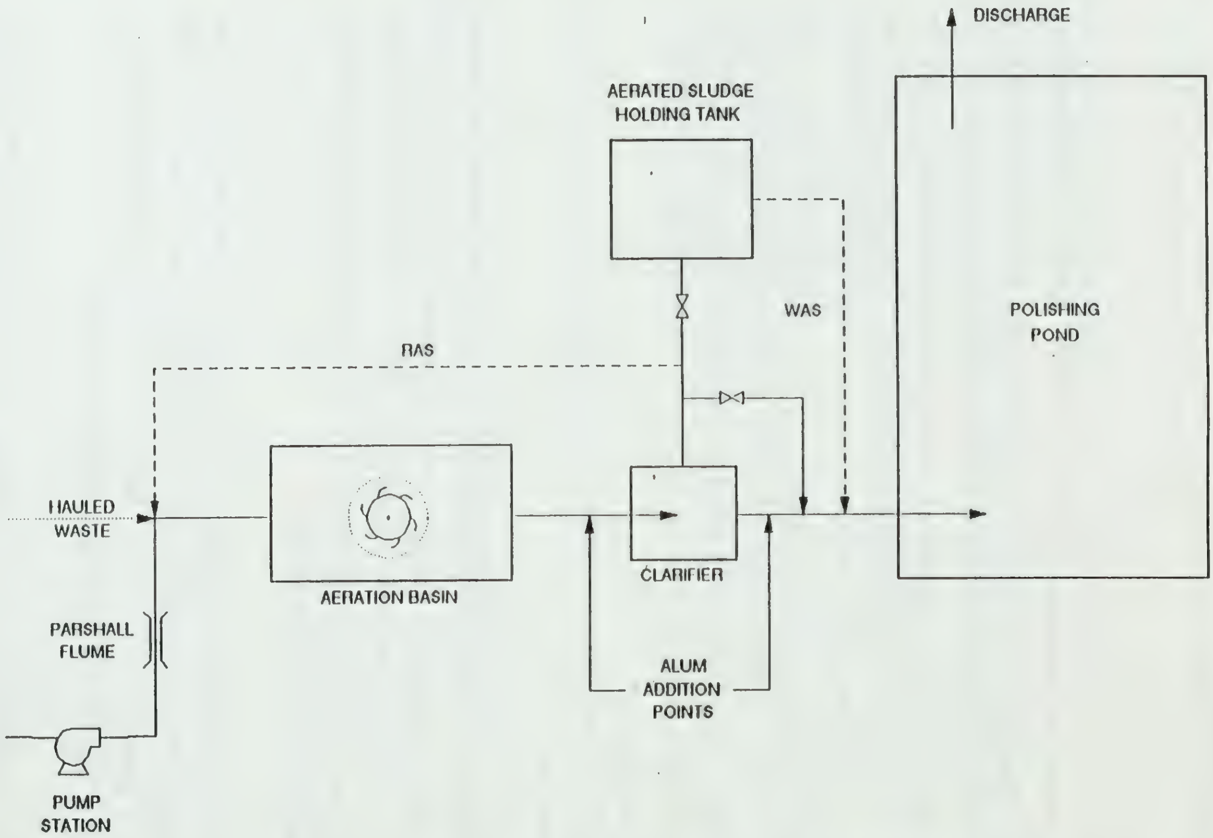


FIGURE A1.1 - SCHEMATIC LAYOUT OF SUTTON WPCP

an approximate depth of 1.5 m.

WAS can be diverted to a 227 m^3 aerated sludge holding tank for stabilization prior to discharge to the lagoon together with the clarified effluent. Alternately, sludge can be wasted directly to the clarifier effluent flow from the return sludge line. Since mid-1991, the aerated holding tank has been used to provide further sludge stabilization prior to discharge of the sludge to the lagoon. Sludge is returned to the aeration tank through a return sludge air lift which is provided by a 18.6 kW blower. This blower also supplies air to the sludge holding tank through course bubble diffusers.

Table A1.1 and Table A1.2 summarize the design criteria and effluent limits respectively for the Sutton WPCP. The Sutton WPCP currently has no effluent limits specified in its C of A; hence, under Policies 08-01 and 08-04, annual average limits of 25 mg/L BOD_5 and 25 mg/L TSS and monthly average limits of 1.0 mg/L TP apply. There are no effluent nitrogen limits for the Sutton WPCP.

TABLE A1.1 DESIGN CRITERIA FOR SUTTON WPCP	
DESIGN CAPACITY (m^3/d)	2046
YEAR OF START-UP	1981
PRELIM.TREATMENT	
Screening	No
Grit Removal	No
Comminutor	No
AERATION	
Design HRT (hrs)	24
Operating SRT	
summer (d)	70
winter (d)	150
Design Organic Loading	
(kg/1000 m^3/d)	131.7*
AERATION METHOD	
	Low Speed Surface Aeration
kW	18.6
kW/1000 m^3 tankage	9.1
CLARIFICATION	
Number	ONE
SOR @ Design Flow (m^3/m^2d)	17.0
SOR @ Peak Flow (m^3/m^2d)	
RAS Ratio (% of flow)	Up to 125
Surface Skimmer	NO
PHOSPHORUS REMOVAL	
Chemical	Alum
Addition Points	Aeration effluent Clarifier effluent
LAGOONS	
No. of Cells	ONE
Surface Area (ha/1000 m^3/d)	3.2
HRT @ Design Flow (d)	44
SLUDGE HOLDING TANK	YES

* Based on 1990 [BODinf]

TABLE A1.2

EFFLUENT LIMITS FOR SUTTON WPCP

Parameter	Basis	Concentration (mg/L)	Loading (kg/d)
BOD_5	Annual Average	25.	NA
TSS	Annual Average	25.	NA
TP	Monthly Average	1.0	NA
Total NH_4-N	NA	NA	NA

NA - Not Applicable

Operation and Performance

Raw sewage, clarifier effluent and lagoon effluent are sampled twice per month and submitted to the MOE for analysis. MLSS is sampled once per month. All samples are grab samples. Routine plant monitoring of phosphorus, solids and ammonia are done on a daily basis.

Table A1.3 summarizes the operating conditions and performance of the Sutton WPCP for 1989 and 1990. The design HRT for the aeration tank is 24 hours. The operating HRT was about 33 hours in both 1989 and 1990. The 1989 and 1990 estimated operating SRTs are both below the design SRT of 70 days in the summer and 150 days in the winter. Design organic loading for Sutton is 132 kg/1000 m^3.d. Actual organic loading for 1989 and 1990 was 106 and 97 kg/1000 m^3.d, respectively.

The clarifier SORs are both below the design SOR of 17 m^3/m^2.d based on average day flow, and peak day SORs for 1989 and 1990 were 23 m^3/m^2.d. The design HRT for the lagoon is 44 days. In 1989 and 1990, the HRTs were 81 and 79 days, respectively. These long retention times may be attributed to an average day flow which is only 70 percent of the average day design flow.

Table A1.4 summarizes the secondary and lagoon effluent quality on an annual average basis for the years 1981 through 1990. Figures A1.2 through A1.6 graphically compare secondary and lagoon effluent on a monthly basis for this time period.

There are no industrial dischargers to the treatment plant; however, hauled wastes from holding tanks are discharged directly to the inlet line of the aeration tank. In 1990, the amount of hauled sewage received was 5000 m^3. No data are available to estimate the effect of hauled wastes on plant loading.

MLSS levels are maintained at 2500 to 3000 mg/L in the summer and 4000 to 5000 mg/L in the winter. In 1988, the aeration basin was out of commission for three days to remove

TABLE: A1.3 Current Operating Conditions

Sutton WPCP

YEAR MONTHS (if LT full year)	1989	1990 (Jan-Jul)
AVERAGE DAY FLOW (m3)	1451	1489
MAXIMUM DAY FLOW (m3) EST.	** 2786	** 2859
RAW SEWAGE:		
AVG. BOD INF. (mg/L)	149.0	133.4
AVG. TSS INF. (mg/L)	137.1	149.0
AVG. TKN INF. (mg/L)	42.4	38.2
AVG. TP INF. (mg/L)	7.0	5.9
AERATION		
OPERATING HRT (hrs)	33.8	33.0
OPERATING SRT (d)	54	58
F/M	0.04	0.03
ORGANIC LOADING (kg/1000m3(AERAT.)*d)	106	97
CLARIFICATION		
OPERATING HRT (hrs)	5.3	5.1
SOR @ AVERAGE FLOW (m^3/m^2 d)	12.1	12.4
SOR @ PEAK DAY FLOW (m^3/m^2 d)	** 23.2	** 23.8
SECONDARY EFFLUENT:		
AVG. BOD (mg/L)	14.9	14.1
STD.DEV.	10.4	9.5
AVG. TSS (mg/L)	9.2	9.7
STD.DEV.	3.9	1.7
AVG. TKN (mg/L)	5.3	2.8
STD.DEV.	6.3	2.1
AVG. NH3-N (mg/L)	4.0	1.7
STD.DEV.	6.1	1.8
AVG. NO(T)-N (mg/L)	N/D	N/D
STD.DEV.	N/D	N/D
AVG. TP (mg/L)	0.2	0.3
STD.DEV.	0.1	0.2
LAGOON (receiving sludge)		
OPERATING HRT (d)	81	79
LAGOON EFFLUENT:		
AVG. BOD (mg/L)	6.2	5.8
STD.DEV.	6.4	2.5
AVG. TSS (mg/L)	9.3	8.9
STD.DEV.	10.5	5.6
AVG. TKN (mg/L)	7.8	10.7
STD.DEV.	5.8	6.4
AVG. NH3-N (mg/L)	6.9	8.7
STD.DEV.	4.6	5.8
AVG. NO(T)-N (mg/L)	N/D	N/D
STD.DEV.	N/D	N/D
AVG. TP (mg/L)	0.6	0.4
STD.DEV.	0.7	0.3
SLUDGE		
ESTIMATED ANNUAL SLUDGE LOADING ON LAGOON (kg/m3 of lagoon) receiving sludge	0.5	0.5
ESTIMATED SLUDGE PRODUCED (kg/d)	170.2	155.4
TOTAL ESTIMATED SLUDGE DISCHARGED TO LAGOON (tonnes) (since start-up)	642	

** PF based on 1991 data to estimate peak day flow

TABLE: A1.4

PERFORMANCE OF SUTTON WPCP

(1981-1990)

	BOD		SS		TP		NH3		TKN	
	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L
1981 *										
AVERAGE	10.54	7.53	39.92	11.80	1.75	0.45	0.26	0.79	2.94	3.08
STD DEV	1.54	2.13	5.92	4.20	0.32	0.14	0.02	0.27	0.44	0.18
1982										
AVERAGE	8.49	4.85	17.48	8.85	1.20	0.90	0.54	1.02	2.74	3.07
STD DEV	6.71	2.13	14.65	4.87	0.58	0.65	1.03	0.54	1.67	0.64
1983										
AVERAGE	5.74	3.25	8.98	3.10	0.39	0.13	1.20	1.85	2.48	4.50
STD DEV	3.06	3.80	4.35	5.00	0.08	0.19	2.93	3.50	2.49	4.75
1984										
AVERAGE	4.34	1.00	7.45	2.50	0.24	0.08	0.43	2.70	1.88	3.40
STD DEV	2.26	0.70	2.90	3.40	0.09	0.04	0.69	3.20	1.72	4.20
1985										
AVERAGE	5.36	2.74	8.63	3.17	0.59	0.21	0.96	1.10	1.89	2.10
STD DEV	2.33	2.49	7.84	1.41	0.44	0.25	1.64	1.18	1.70	1.17
1986										
AVERAGE	7.55	3.08	9.85	4.19	0.57	0.43	0.12	N/D	1.20	N/D
STD DEV	3.20	2.70	4.89	1.65	0.18	0.48	0.13	N/D	0.46	N/D
1987										
AVERAGE	7.36	4.95	8.73	3.97	0.65	0.38	4.24	2.42	5.06	3.45
STD DEV	3.85	2.50	3.20	3.43	0.96	0.33	4.52	2.59	4.85	2.66
1988										
AVERAGE	8.60	4.09	7.89	5.14	0.29	0.21	0.20	0.95	1.36	2.04
STD DEV	5.87	2.53	4.99	3.43	0.25	0.12	0.13	1.14	0.77	1.03
1989										
AVERAGE	14.93	6.21	9.22	9.34	0.23	0.63	3.96	6.90	5.28	7.75
STD DEV	10.40	6.37	3.89	10.47	0.12	0.74	6.13	4.62	6.32	5.76
1990 **										
AVERAGE	14.14	5.84	9.73	8.93	0.30	0.38	1.68	8.65	2.83	10.69
STD DEV	9.50	2.52	1.72	5.56	0.15	0.27	1.81	5.83	2.07	6.44

* Nov.-Dec.   ** Jan.-Oct.

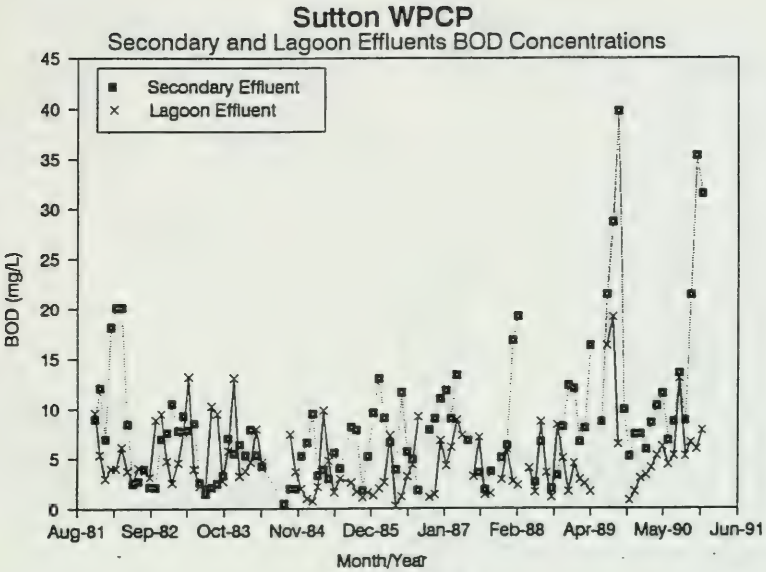


Figure A1.2

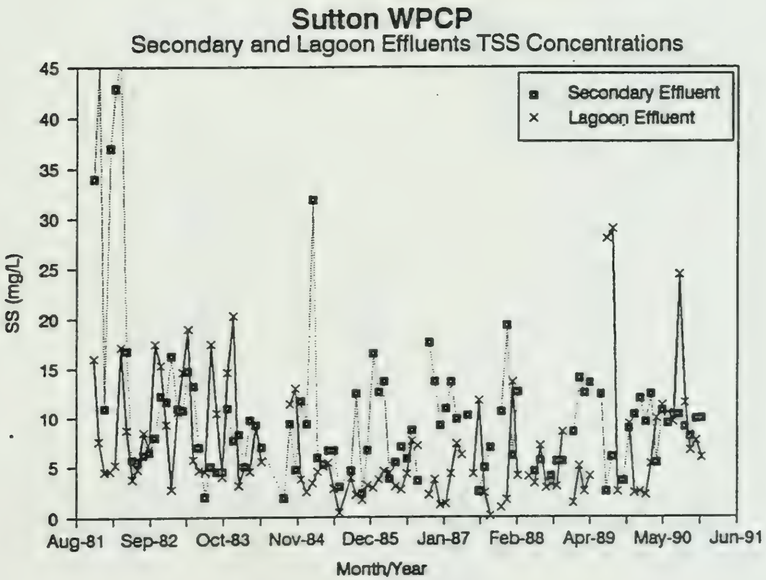


Figure A1.3

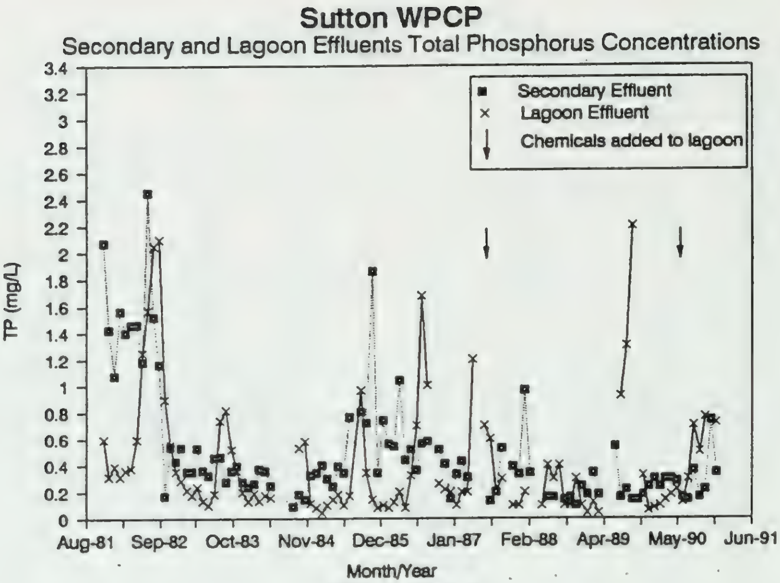


Figure A1.4

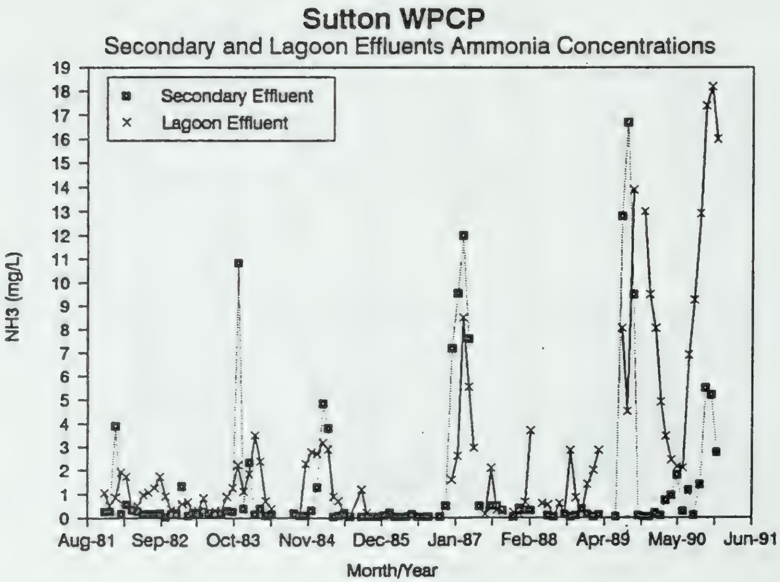
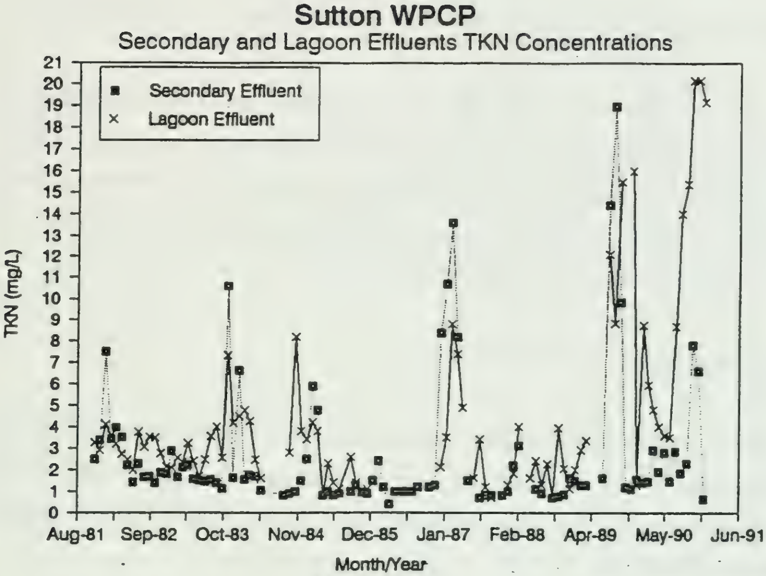


Figure A1.5



Sutton WPCP
Secondary and Lagoon Effluents TKN Concentrations
Secondary Effluent
Lagoon Effluent
TKN (mg/L)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
Aug-81
Sep-82
Oct-83
Nov-84
Dec-85
Jan-87
Feb-88
Apr-89
May-90
Jun-91
Month/Year

Figure A1.6

accumulated grit. During this period, raw sewage was diverted directly to the lagoon.

In the past few years, the phosphorus and ammonia levels have been increasing in the lagoon as evident from Figures A1.4 and A1.5. As a result, Sutton no longer discharges from the lagoon from June through August. To reduce the phosphorus concentration in the lagoon, the plant began batch dosing the lagoon with alum in 1987. Alum is added directly to the clarifier effluent to accomplish this batch dosing.

Sludge Loading and Sludge Accumulation

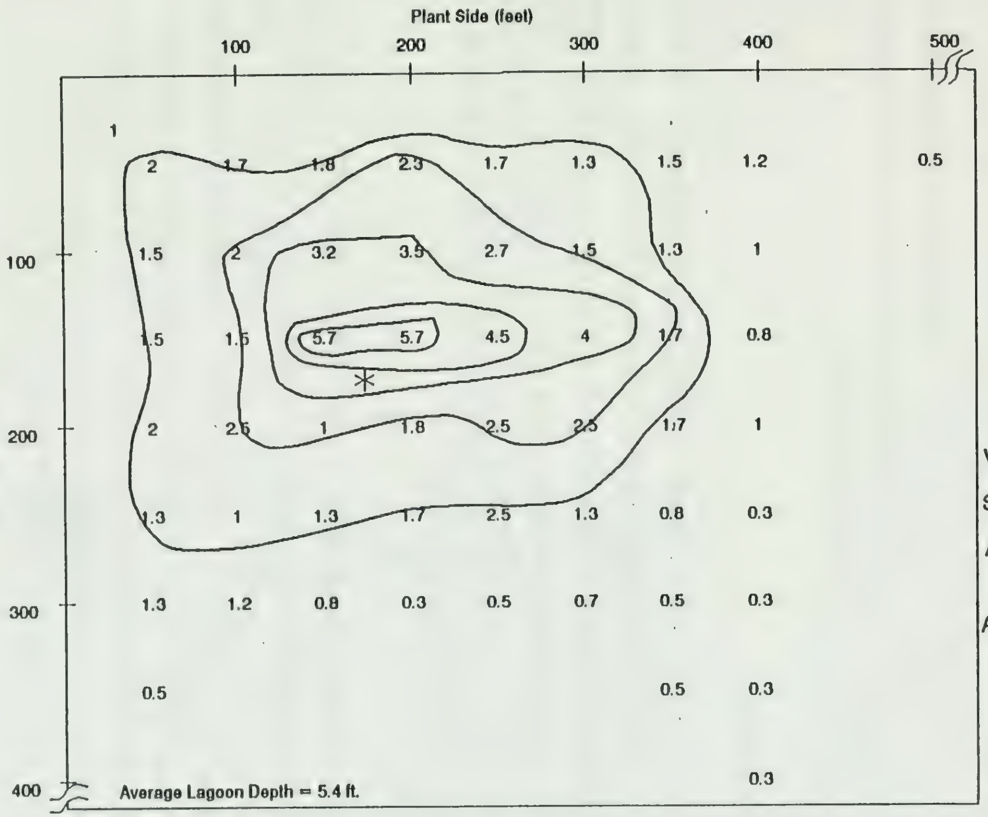
Based on operating conditions for 1989 and 1990, the annual sludge loading on the Sutton WPCP lagoon was 0.5 kg/m^3 of lagoon. The estimated sludge produced for 1989 and 1990 was 170 and 155 kg/d respectively. Approximately 642 tonnes of sludge has been discharged to the lagoon since start-up.

Figure A1.7 illustrates approximate sludge depths in the Sutton lagoon using sludge depth mapping information supplied by the MOE. It is estimated that approximately 3.1 percent of the lagoon volume is filled with sludge. Core sampling indicates an average sludge concentration between about 6 percent and 13 percent solids within the sludge pile.

Figure: A1.7

Sutton WPCP

Lagoon Sludge Depths



Volume of sludge = 3656 m3

Sludge Mass = 642 tonnes

Average sludge concentration = 177 kg/m3

Approx. volume of lagoon filled with sludge = 3.1%

* Intake Pipe Approx. 175 ft. out from plant and van side

A1.2 COLBORNE WPCP

Background

The Sutton process was implemented at the Colborne WPCP in 1984. The original lagoon system was upgraded including the addition of a raw sewage lift station and an extended aeration plant with a rated capacity of 1,375 m^3/d. The only modification to the lagoon at this time was to relocate the influent piping. One stoplog was removed from the lagoon effluent resulting in the lagoon level dropping approximately 0.2 meters. No sludge was removed from the lagoon at the time of the upgrading.

Process Description

A flow schematic of the Colborne WPCP is shown in Figure A1.8. Influent to the treatment plant passes through a manually cleaned bar screen and enters the wet well. From the wet well raw sewage is pumped to a single channel which divides into two parallel grit channels. There are two on-off submersible pumps with a third pump used for standby. All three pumps are 3.7 kW. A comminutor installed in the first grit channel is used for all plant flow. The second grit channel has a bar screen and is used for by-pass purposes only when the comminutor is out-of-service.

From the grit channel, influent passes through a Parshall flume where flow is measured using an ultrasonic sensor. Flow metering inaccuracies have been a concern at the plant and the flow meter was recalibrated in 1991. This recalibration showed a significant overestimate of actual plant flow. A correction factor has been applied to all historic flow data to compensate for the flow meter error. Flow is directed to the parallel aeration tank via three influent ports. Each tank is 8.5 m by 25 m and has a depth of 4.3 m. Air is supplied by two 5.6 kW mechanical surface aerators which have an extended shaft with two impellers. Alum is added to the aeration tank effluent for phosphorus removal.

From the aeration tank, mixed liquor enters the clarifier which has a surface area of 153 m^3

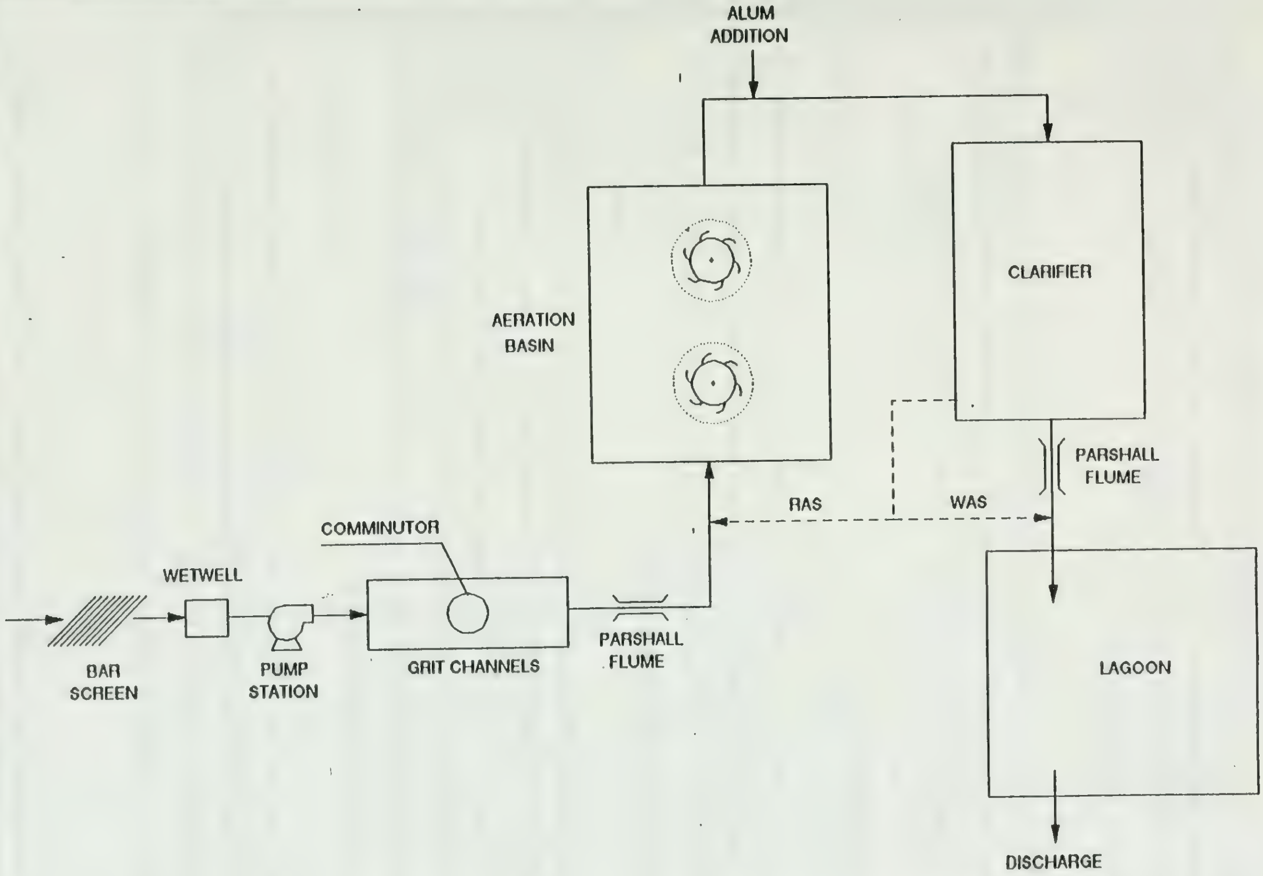


FIGURE A1.8 - SCHEMATIC LAYOUT OF COLBORNE WPCP

and a depth of 3.7 m. Settled sludge is either wasted to the lagoon or returned to the influent channel of the aeration tank. Effluent from the final clarifier passes through a channel with a Parshall flume to measure effluent flow prior to discharge to the lagoon. The lagoon has a total surface area of 3.2 ha.

Table A1.5 and Table A1.6 summarize the design criteria and effluent limits, respectively. The Colborne WPCP has specific limits for BOD, phosphorus and ammonia specified in the Certificate of Approval. The limit for TSS is based on Policies 08-01 and 08-04.

Operation and Performance

Raw sewage, aeration plant effluent and lagoon effluent are sampled twice per month and submitted to the MOE for analysis. MLSS is also sampled twice per month. All samples are manually collected 4 to 5 hour composites. In the spring of 1991, a 24 hour composite sampler was installed at the lagoon effluent.

Table A1.7 summarizes the operating conditions and performance of the Colborne WPCP for 1989 and 1990. The design HRT for the aeration tank is 16 hours. The operating HRT was 28 and 24 hours in 1989 and 1990, respectively based on corrected flows. The 1989 and 1990 operating SRTs are both above the design SRT of 10 to 30 days. The design organic loading at Colborne is 206 kg/1000 m^3.d. Actual organic loadings for 1989 and 1990 were 75 and 99 kg/1000 m^3.d, respectively.

The clarifier SORs are both below the design SOR of 8.9 m^3/m^2.d based on average day flow, and peak day SORs for 1989 and 1990 were below the design value of 31.6 m^3/m^2.d. The design HRT for the lagoon is 33 days. In 1989 and 1990 the HRTs were 67 and 58 days, respectively. Average day flows have been less than 70 percent of the average day design flow.

Table A1.8 summarizes the secondary and lagoon effluent on an annual average basis for the years 1983 through 1991. Figures A1.9 through A1.13 graphically present a

TABLE A15

DESIGN CRITERIA FOR COLBORNE WPCP

DESIGN CAPACITY (m^3/d)	1375 4826 (peak)
YEAR OF START-UP	1983
PRELIM.TREATMENT	
Screening	coarse
Grit Removal	Yes
Comminutor	Yes
AERATION	
Design HRT (hrs)	16
Operating SRT	
summer (d)	10 to 30
winter (d)	per O&M manual
Design Organic Loading ($g/1000\ m^3/d$)	206.1
AERATION METHOD	Low Speed Surface Aeration
kW	11.2
kW/1000 m^3 tankage	12.3
CLARIFICATION	
Number	One
SOR @ Design Flow (m^3/m^2d)	8.9
SOR @ Peak Flow (m^3/m^2d)	31.6
RAS Ratio (% of flow)	100
Surface Skimmer	No
PHOSPHORUS REMOVAL	
Chemical	Alum
Addition Points	aeration & clarifier effluents
LAGOONS	
No. of Cells	One
Surface Area ($ha/1000\ m^3/d$)	2.3
HRT @ Design Flow (d)	33
SLUDGE HOLDING TANK	No

N/A - Not Applicable

TABLE A1.6

EFFLUENT LIMITS FOR COLBORNE WPCP

Parameter	Basis	Concentration (mg/L)	Loading (kg/d)
BOD	Annual Average	15.	NA
TSS	Annual Average	25.	NA
TP	Monthly Average	1.0	NA
Total NH_4-N	May 15 - Oct. 1	2.00	NA
	T > 15°C	3.00	NA
	T > 10°C	4.50	NA
	T > 5°C	6.80	NA

NA = Not Applicable

TABLE: A1.7 Current Operating Conditions

Colborne WPCP

YEAR	1989	1990
AVERAGE DAY FLOW (m3)	* 789	* 912
MAXIMUM DAY FLOW (m3) EST.	** 1728	** 1998
RAW SEWAGE:		
AVG. BOD INF. (mg/L)	87.4	99.0
AVG. TSS INF. (mg/L)	154.9	116.0
AVG. TKN INF. (mg/L)	N/D	30.0
AVG. TP INF. (mg/L)	4.5	4.9
AERATION		
OPERATING HRT (hrs)	27.8	24.0
OPERATING SRT (d)	73	56
F/M	0.03	0.03
ORGANIC LOADING		
(kg/1000m3(AERAT.)*d)	75	99
CLARIFICATION		
OPERATING HRT (hrs)	17.2	14.8
SOR @ AVERAGE FLOW (m^3/m^2 d)	5.2	6.0
SOR @ PEAK DAY FLOW (m^3/m^2 d)	** 11.33	** 13.1
SECONDARY EFFLUENT:		
AVG. BOD (mg/L)	5.4	4.9
STD.DEV.	3.0	1.2
AVG. TSS (mg/L)	13.1	9.2
STD.DEV.	7.9	2.8
AVG. TKN (mg/L)	1.0	0.8
STD.DEV.	0.4	0.2
AVG. NH3-N (mg/L)	0.1	0.1
STD.DEV.	0.03	0.03
AVG. NO(T)-N (mg/L)	14.7	12.8
STD.DEV.	2.6	1.6
AVG. TP (mg/L)	0.7	0.5
STD.DEV.	0.2	0.2
LAGOON (receiving sludge)		
OPERATING HRT (d)	67	58
LAGOON EFFLUENT:		
AVG. BOD (mg/L)	3.2	3.7
STD.DEV.	2.4	5.3
AVG. TSS (mg/L)	4.3	6.8
STD.DEV.	2.3	12.3
AVG. TKN (mg/L)	3.3	1.6
STD.DEV.	2.6	1.0
AVG. NH3-N (mg/L)	2.0	4.0
STD.DEV.	2.4	0.7
AVG. NO(T)-N (mg/L)	4.0	2.9
STD.DEV.	3.4	2.7
AVG. TP (mg/L)	0.5	0.3
STD.DEV.	0.3	0.2
SLUDGE		
ESTIMATED ANNUAL SLUDGE LOADING ON LAGOON (kg/m3 of lagoon) receiving sludge	0.4	0.5
ESTIMATED SLUDGE PRODUCED (kg/d)	56.6	75.1
TOTAL ESTIMATED SLUDGE DISCHARGED TO LAGOON (tonnes) (since start-up)	216	

* correction factor for flow recorder error included

** P.F. based on 1991 data to estimate peak day flow

TABLE: A1.8

PERFORMANCE OF COLBORNE WPCP

(1983–1991)

	BOD		SS		TP		NH3		TKN	
	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L
1983										
AVERAGE	7.00	6.20	8.90	11.00	1.72	1.29	0.29	3.84	1.65	6.48
STD DEV	2.88	5.15	3.41	11.19	0.70	0.71	0.10	1.56	0.54	2.17
1984										
AVERAGE	6.53	2.70	9.93	5.07	1.53	0.78	0.42	1.68	1.62	2.97
STD DEV	2.92	2.02	3.92	3.78	0.71	0.54	0.43	1.90	0.59	2.22
1985										
AVERAGE	4.50	1.80	11.33	4.39	0.65	0.45	0.21	0.25	2.32	1.18
STD DEV	2.02	0.93	6.99	5.27	0.26	0.15	0.12	0.15	2.65	0.29
1986										
AVERAGE	5.54	3.22	10.77	3.59	0.53	0.25	0.15	0.23	1.10	1.18
STD DEV	2.01	2.66	4.72	4.45	0.14	0.12	0.21	0.23	0.62	0.53
1987										
AVERAGE	7.23	4.58	15.73	5.70	0.68	0.18	0.32	0.71	1.45	1.90
STD DEV	1.23	2.81	3.22	5.85	0.15	0.04	0.45	0.28	0.46	0.25
1988										
AVERAGE	6.59	3.76	10.60	7.65	0.73	0.31	1.37	1.89	2.28	3.09
STD DEV	5.33	3.12	7.68	6.63	0.25	0.16	3.81	2.69	4.45	2.86
1989										
AVERAGE	5.39	3.19	13.09	4.26	0.65	0.48	0.06	N/D	0.97	N/D
STD DEV	3.01	2.40	7.89	2.25	0.23	0.28	0.03	N/D	0.37	N/D
1990										
AVERAGE	4.91	3.68	9.19	6.77	0.45	0.27	0.06	0.40	0.84	1.57
STD DEV	1.17	5.26	2.76	12.27	0.16	0.17	0.03	0.68	0.24	0.96
1991										
AVERAGE	3.99	5.17	9.32	11.33	0.35	0.25	0.07	0.89	0.85	2.18
STD DEV	2.66	3.44	6.37	11.75	0.12	0.12	0.03	1.09	0.22	1.45

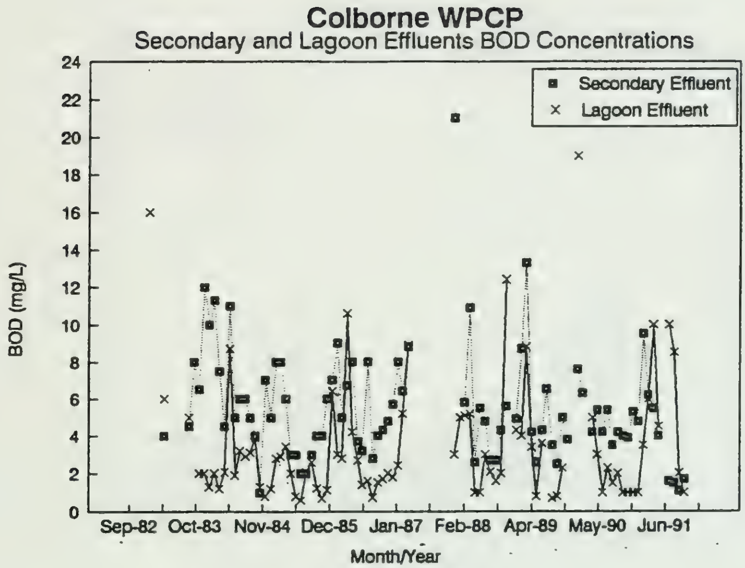


Figure A1.9

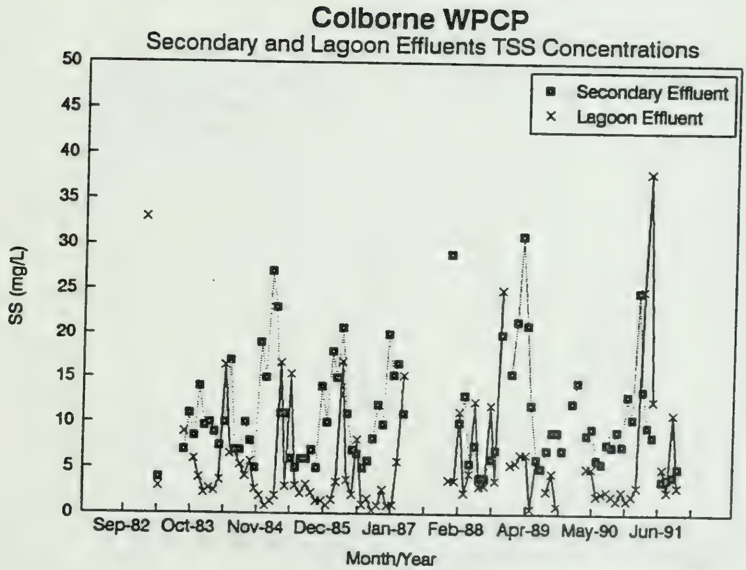


Figure A1.10

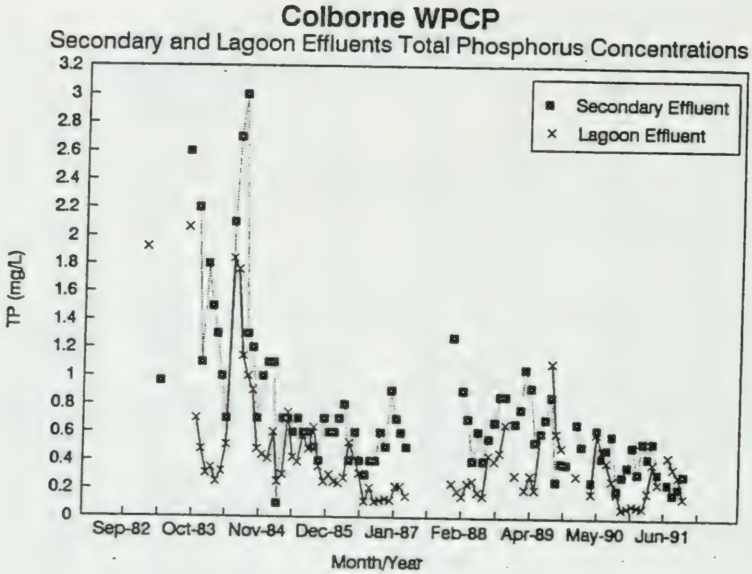


Figure A1.11

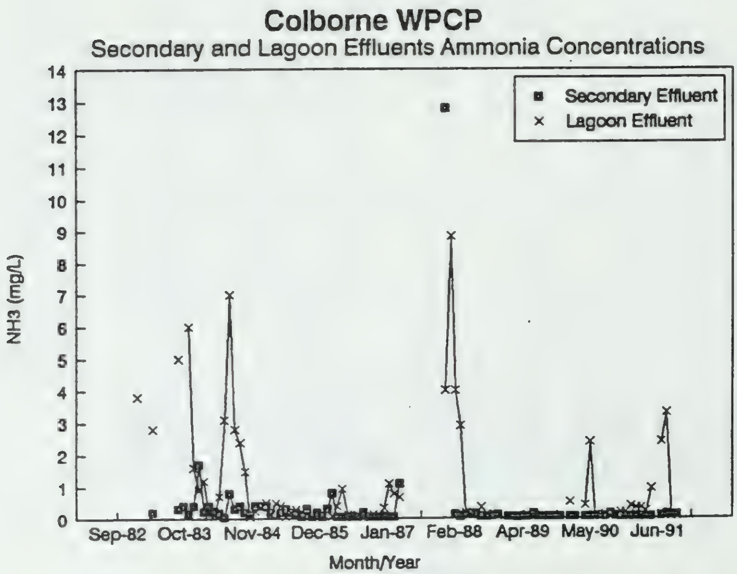


Figure A1.12

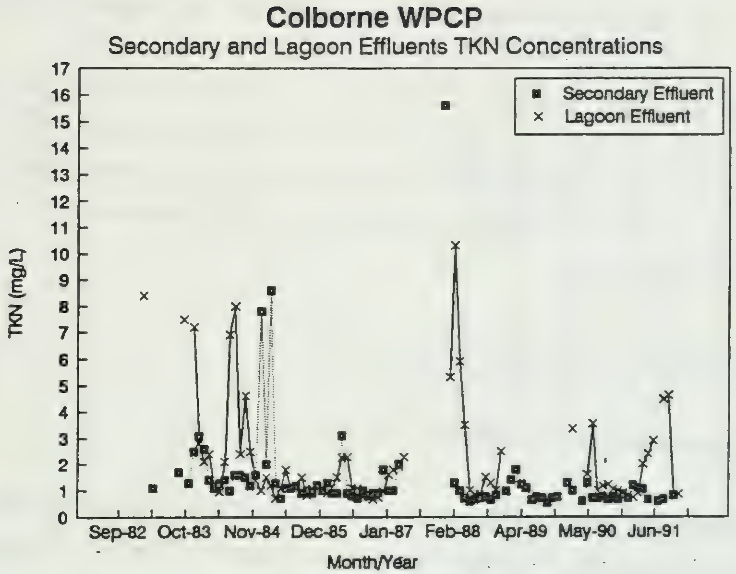

Colborne WPCP
Secondary and Lagoon Effluents TKN Concentrations
TKN (mg/L)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Secondary Effluent
Lagoon Effluent
Sep-82
Oct-83
Nov-84
Dec-85
Jan-87
Feb-88
Apr-89
May-90
Jun-91
Month/Year

Figure A1.13

comparison of secondary and lagoon effluent on a monthly basis for the same time period.

MLSS levels are maintained at 3500 to 4000 mg/L in the summer and 4500 to 5000 mg/L in the winter. Sludge is wasted twice per week for two hours to the lagoon by diverting the RAS. In the winter, sludge is wasted once per week or once every two weeks. Volumes of waste sludge are not recorded and sludge is not sampled. SRT was estimated based on typical yields and loading rates.

There are no industrial discharges to the Colborne WPCP.

Sludge Loading and Sludge Accumulation

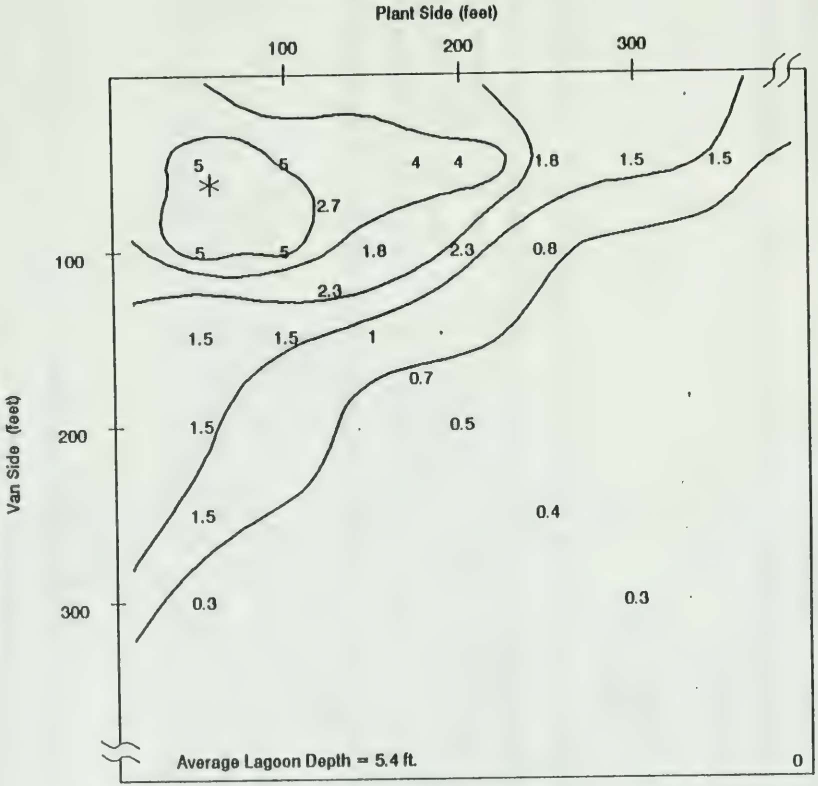
Based on operating conditions for 1989 and 1990, the annual sludge loading on the Colborne WPCP lagoon was 0.4 kg/m^3 of lagoon. The estimated sludge produced for 1989 and 1990 was 57 and 75 kg/d, respectively. Approximately 216 tonnes of sludge has been discharged to the lagoon since start-up.

Figure A1.14 illustrates approximate sludge depths in the Colborne lagoon using sludge depth mapping information supplied by the MOE. It is estimated that approximately 4.7 percent of the lagoon volume is filled with sludge. Core sampling indicates an average sludge concentration of about between 9 percent and 14 percent solids within the sludge pile.

Figure: A1.14

Colborne WPCP

Lagoon Sludge Depths



✳ Intake Pipe Approx. 50 ft. out from plant and van side

Volume of sludge = 2458 m3

Sludge mass = 216 tonnes

Average sludge concentration = 88 kg/m3

Approx. volume of lagoon filled with sludge = 4.7 %

A1.3 TOTTENHAM WPCP

Background

The original Tottenham lagoons date back to 1971 when they were used as seasonal discharge lagoons. During this time, the lagoons were batch treated with alum prior to discharge for phosphorus removal. In 1986, concerns regarding receiving water quality identified the need to upgrade the facility. The Sutton process was chosen and commissioned in 1988. This upgrade included the extended aeration plant, a flow splitter box at the lagoons, and excavation of a 75 by 75 meter square to a depth of 0.75 m at the inlet area of lagoons 1 and 2 for sludge disposal. At this time the lagoons were dewatered. Minimal accumulation of sludge were found and these solids were redistributed over the bottom of the lagoons.

Process Description

A flow schematic of the Tottenham WPCP is shown in Figure A1.15. Influent is pumped to the treatment facility from an off-site pumping station and is split between two aeration cells. There are no prescreening or grit removal facilities. Plant flow is measured by means of a magnetic flowmeter on the pump station discharge. Each lined aeration cell has a volume of 1127 m^3. Air is supplied by a 18.6 kW low speed surface aerator with a bottom mixer. Aeration effluent is directed to a single circular clarifier after alum addition.

The clarifier has a diameter of 16.85 m and a depth of 3.6 m. RAS flows by gravity to a sump where two pumps (one service, one standby) pump the sludge to the raw sewage/RAS mixing chamber. From this mixing chamber, WAS can be diverted for discharge to the lagoons.

Clarifier effluent and WAS is split between lagoon #1 and #2. Flow from lagoon #1 enters lagoon #4 and finally lagoon #3 for final discharge. Flow from lagoon #2 enters lagoon #3 prior to final discharge.

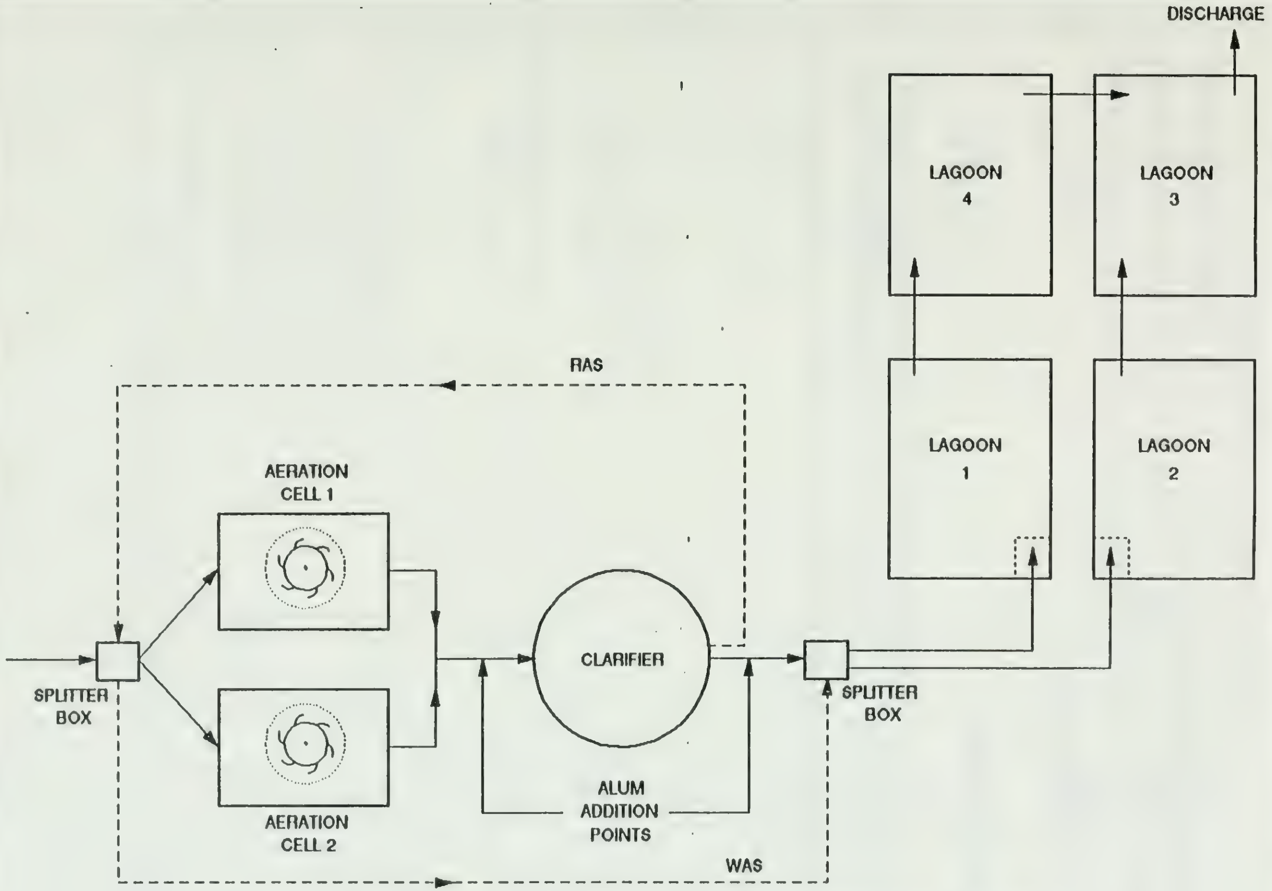


FIGURE A1.15 - SCHEMATIC LAYOUT OF TOTTENHAM WPCP

Table A1.9 and Table A1.10 summarize the design criteria and effluent limits, respectively. The Tottenham WPCP has annual limits for BOD, TSS, and phosphorus outlined in the Certificate of Approval. The Certificate of Approval also specifies ammonia effluent limits which vary from month to month.

Operation and Performance

Raw sewage, clarifier and lagoon effluents are sampled every two weeks and submitted to the MOE for analysis. Raw sewage is a 24 hour composite sample and all others are grab samples. Extensive in-plant monitoring and analysis is conducted for process control.

TABLE A1.9 DESIGN CRITERIA FOR TOTTENHAM WPCP	
DESIGN CAPACITY (m^3/d)	2257 7960(peak)
YEAR OF START-UP	1988
PRELIM.TREATMENT	
Screening	coarse
Grit Removal	No
Comminutor	No
AERATION	
Design HRT (hrs)	24
Operating SRT	
summer (d)	NA
winter (d)	NA
Design Organic Loading (kg/1000 m^3/d)	170.0
AERATION METHOD	low speed surface aeration
kW	37.2
kW/1000 m^3 tankage	16.5
CLARIFICATION	
- Number	One
SOR @ Design Flow (m^3/m^2d)	10.1
SOR @ Peak Flow (m^3/m^2d)	35.0
RAS Ratio (% of flow)	100 to 200
Surface Skimmer	No
PHOSPHORUS REMOVAL	
Chemical	Alum
Addition Points	aeration effluent
LAGOONS	
No. of Cells	Four
Surface Area (ha/1000 m^3/d)	5.0
HRT @ Design Flow (d)	75
LAGOONS RECEIVING SLUDGE	
Surface Area (ha)	5.0
HRT @ Design Flow (d)	55
SLUDGE HOLDING TANK	

TABLE A1.10

EFFLUENT LIMITS FOR TOTTENHAM WPCP

Parameter	Basis	Concentration (mg/L)	Loading (kg/y)
BOD_5	Annual Average	7.0	NA
TSS	Annual Average	15.0	NA
TP	Monthly Average	1.00	820
Total NH_4-N	4-month average Dec-Mar	4.70	NA
	2-month average June-Sept	1.60	NA
	4-month average Apr-May	0.70	NA
	2-month average Oct-Nov	1.60	NA

NA = Not Applicable

Table A1.11 summarizes the operating conditions at the Tottenham WPCP for 1990 and 1991 (Jan-Sep). The design HRT for the aeration tank is 24 hours. The operating HRT was 35 and 37 hours in 1990 and 1991 respectively. The design organic loading at Tottenham is 170 kg/1000 m^3.d. Actual organic loading for 1990 and 1991 was 139 and 84 kg/1000 m^3.d, respectively.

The clarifier SORs are both below the design SOR of 10.1 m^3/m^2.d based on average day flow, and peak day SORs for 1990 and 1991 were below the design value of 35.0 m^3/m^2.d. The design HRT for the lagoon is 75 days. In 1990 and 1991, the HRTs were 49 and 52 days respectively.

Table A1.12 summarizes the secondary and lagoon effluent quality for the years 1988 through 1991. Figures A1.16 through A1.20 graphically compare secondary and lagoon effluent on a monthly basis for this time period.

MLSS levels are maintained at 4000 mg/L in the summer and 5500 mg/L in the winter. Sludge is wasted on Mondays and Fridays. RAS concentration is measured and WAS flow is estimated based on the pump run timer.

Grit accumulation is a problem in the splitter box, since there are no screening or grit removal facilities prior to the aeration tanks.

There are no major industrial discharges to the Tottenham WPCP. Septic haulers discharging to the sewers have historically caused problems such as odour and low DO at the treatment plant.

Sludge Loading and Sludge Accumulation

Based on operating conditions for 1990 and 1991, the annual sludge loading on the Tottenham WPCP lagoons was 1.3 and 0.8 kg/m^3 of lagoon based on the volume of lagoons

TABLE: A1.11

Current Operating Conditions

Tottenham WPCP

YEAR MONTHS	1990	1991 (Jan-Sep)
AVERAGE DAY FLOW (m3)	1545	1475
MAXIMUM DAY FLOW (m3)	4510	4663
RAW SEWAGE:		
AVG. BOD INF. (mg/L)	203.0	128.5
AVG. TSS INF. (mg/L)	324.0	191.1
AVG. TKN INF. (mg/L)	34.2	27.8
AVG. TP INF. (mg/L)	10.0	5.6
AERATION		
OPERATING HRT (hrs)	35.06	36.7
OPERATING SRT (d)	45.17	75
F/M	0.04	0.03
ORGANIC LOADING (kg/1000m3(AERAT.)*d)	139	84
CLARIFICATION		
OPERATING HRT (hrs)	12.5	13.1
SOR @ AVERAGE FLOW (m^3/m^2 d)	6.9	6.6
SOR @ PEAK DAY FLOW (m^3/m^2 d)	20.2	20.9
SECONDARY EFFLUENT:		
AVG. BOD (mg/L)	7.3	4.6
STD.DEV.	4.2	5.4
AVG. TSS (mg/L)	14.1	6.0
STD.DEV.	9.9	4.4
AVG. TKN (mg/L)	1.1	0.8
STD.DEV.	0.3	0.2
AVG. NH3-N (mg/L)	N/D	0.1
STD.DEV.	N/D	0.1
AVG. NO(T)-N (mg/L)	21.3	21.1
STD.DEV.	5.9	2.8
AVG. TP (mg/L)	1.4	0.6
STD.DEV.	0.7	0.3
LAGOON (receiving sludge)		
OPERATING HRT (d)	49	52
LAGOON EFFLUENT:		
AVG. BOD (mg/L)	3.2	2.5
STD.DEV.	3.4	2.0
AVG. TSS (mg/L)	6.5	3.3
STD.DEV.	7.9	2.6
AVG. TKN (mg/L)	1.5	1.4
STD.DEV.	0.5	0.4
AVG. NH3-N (mg/L)	0.3	0.2
STD.DEV.	0.3	0.2
AVG. NO(T)-N (mg/L)	3.3	2.8
STD.DEV.	3.7	3.2
AVG. TP (mg/L)	0.5	0.3
STD.DEV.	0.2	0.1
SLUDGE		
ESTIMATED ANNUAL SLUDGE LOADING ON LAGOON (kg/m3 of lagoon) receiving sludge	1.3	0.8
ESTIMATED SLUDGE PRODUCED (kg/d)	274.8	165.9
TOTAL ESTIMATED SLUDGE DISCHARGED TO LAGOON (tonnes) (since start-up)	322	

TABLE: A1.12

PERFORMANCE OF TOTTENHAM WPCP

(1988–1991)

	BOD		SS		TP		NH3		TKN	
	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L	SEC. mg/L	LAG. mg/L
1988*										
AVERAGE	6.05	7.56	14.63	8.76	6.12	0.36	N/D	1.45	1.10	3.54
STD DEV	1.65	4.69	7.71	6.91	2.91	0.35	N/D	0.80	0.31	2.19
1989										
AVERAGE	7.96	4.32	14.73	8.63	1.34	0.29	0.45	0.46	1.17	1.73
STD DEV	5.15	3.16	7.37	8.87	0.78	0.06	0.00	0.31	0.55	0.51
1990										
AVERAGE	7.30	3.20	14.10	6.50	1.40	0.50	0.03	0.30	1.06	1.50
STD DEV	4.20	3.40	9,90	7.90	0.70	0.20	0,01	0.30	0.30	0.50
1991**										
AVERAGE	4.60	2.50	6.00	3.30	0.60	0.30	0.10	0.20	0.80	1.40
STD DEV	5,40	2,00	4,40	2,60	0,30	0,10	0,10	0,20	0,20	0.40

* Apr–Dec
** Jan–Sep

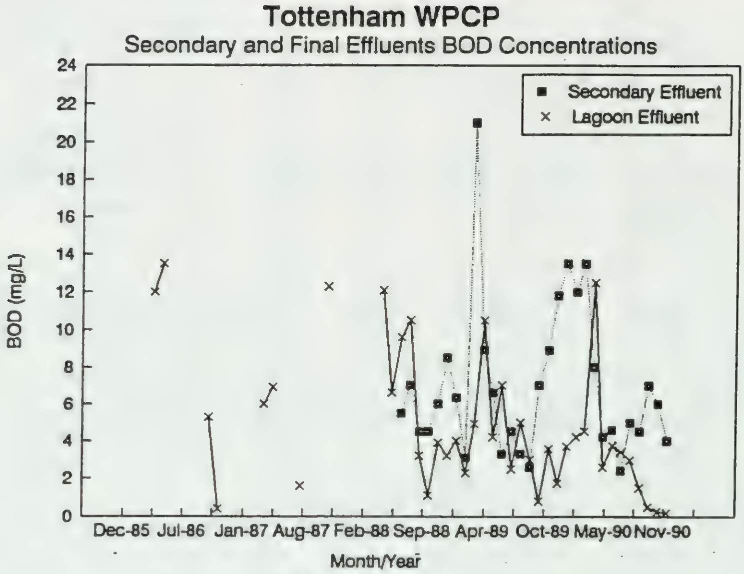


Figure A1.16

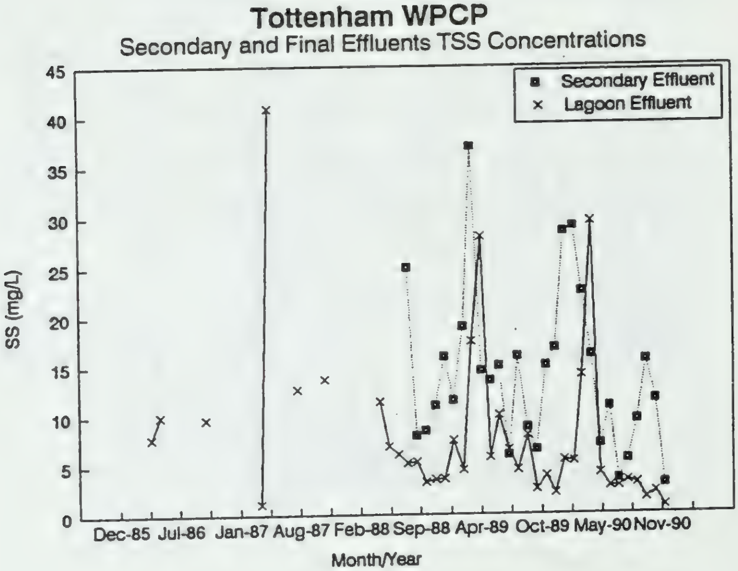


Figure A1.17

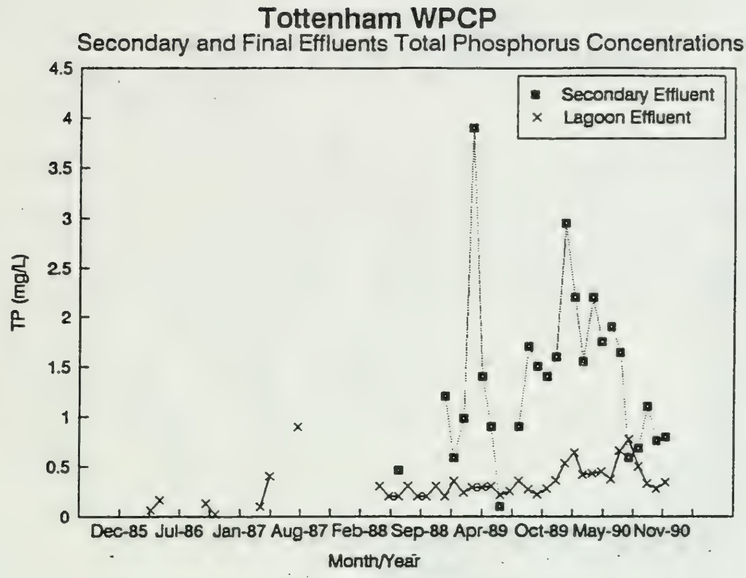


Figure A1.18

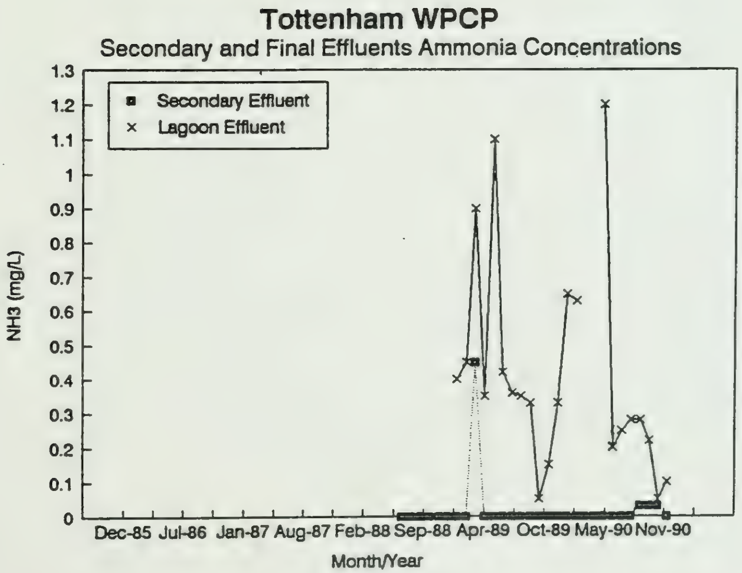


Figure A1.19

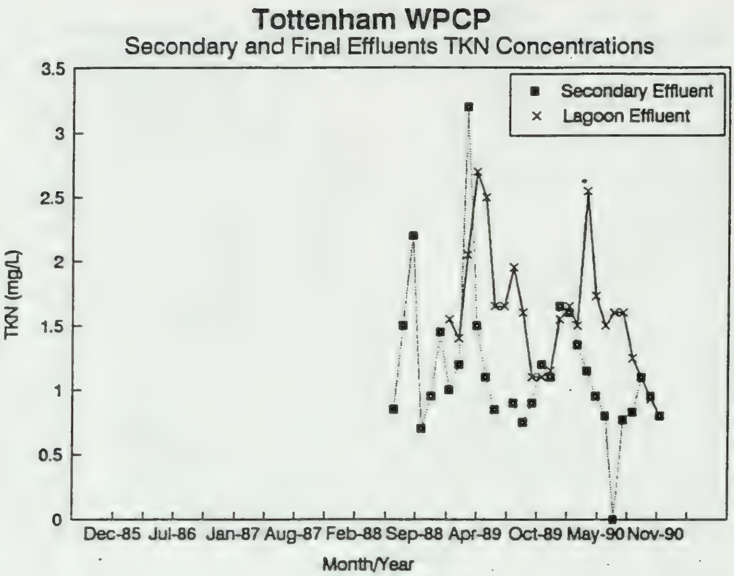

Tottenham WPCP
Secondary and Final Effluents TKN Concentrations
Secondary Effluent
Lagoon Effluent
TKN (mg/L)
3.5
3
2.5
2
1.5
1
0.5
0
Dec-85 Jul-86 Jan-87 Aug-87 Feb-88 Sep-88 Apr-89 Oct-89 May-90 Nov-90
Month/Year

Figure A1.20

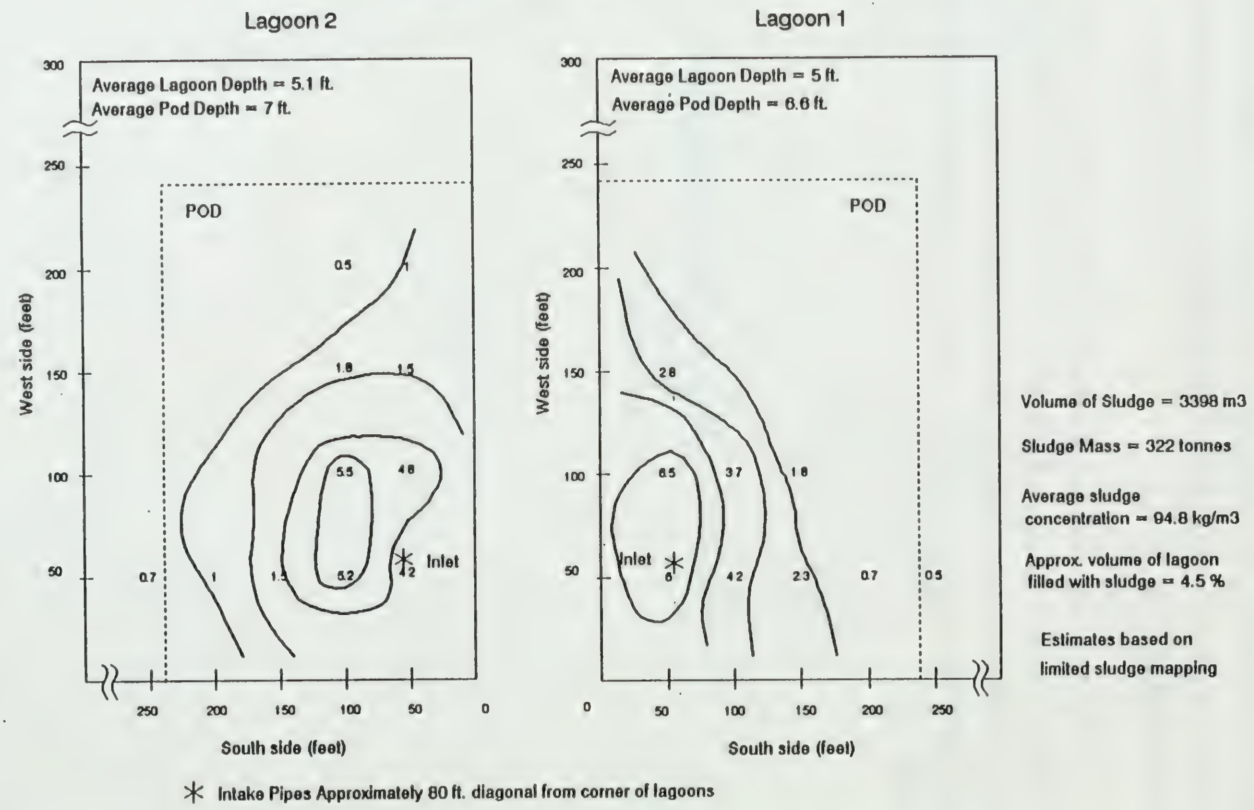
#1 and #2 which receive sludge. The estimated sludge produced for 1990 and 1991 was 275 and 166 kg/d, respectively. Approximately 322 tonnes of sludge has been discharged to the lagoon since start-up.

Figure A1.21 illustrates approximate sludge depths in the Tottenham lagoons using sludge depth mapping information supplied by the MOE. It is estimated that approximately 4.5 percent of the lagoon volume (lagoon #1 and #2) is filled with sludge. Core samling indicates an average sludge concentration of between 2.5 and 4.0 percent solids within the sludge pile.

Figure: A1.21

Tottenham WPCP

Lagoon Sludge Depths



A1.4 COOKSTOWN WPCP

Background

The Cookstown WPCP was commisioned as a Sutton Process plant in 1988. Prior to this, the town had been serviced by septic tanks. The new facility has a design capacity of 825 m^3.

Process Description

A flow schematic of the Cookstown WPCP is shown in Figure A1.22. Raw sewage enters the plant pumping station and is pumped at 30.5 l/s to the headworks. Preliminary treatment consists of parallel grit channels followed by a bar screen and comminutor.

From the comminutor, flow is directed into an annular aeration basin. The aeration basin has a volume of 960 m^3. Air is supplied by five 2.5 kW AIRE-O_2 aerators. Alum is added for phosphorus removal to the aeration tank effluent prior to clarification.

The circular clarifier makes up the inside area of the annular aeration tank. The clarifier has a diameter of 10 m and a depth of 3.6 m. Sludge is either pumped back to the aeration tank influent or wasted to either of the lagoons through a series of valves. Clarified effluent is split between two lagoons each having a surface area of 2.47 ha and an approximate depth of 2.7 m.

Table A1.13 and Table A1.14 summarize the design criteria and effluent limits, respectively. The Cookstown WPCP has specific limits for BOD, TSS and phosphorus outlined in the Certificate of Approval. The Certificate of Approval also outlines detailed ammonia effluent limits for specific months, as well as limits on the effluent discharge rate. No discharge is allowed from June 1 through September 30. During this period, there is total retention of the treated effluent in the lagoons.

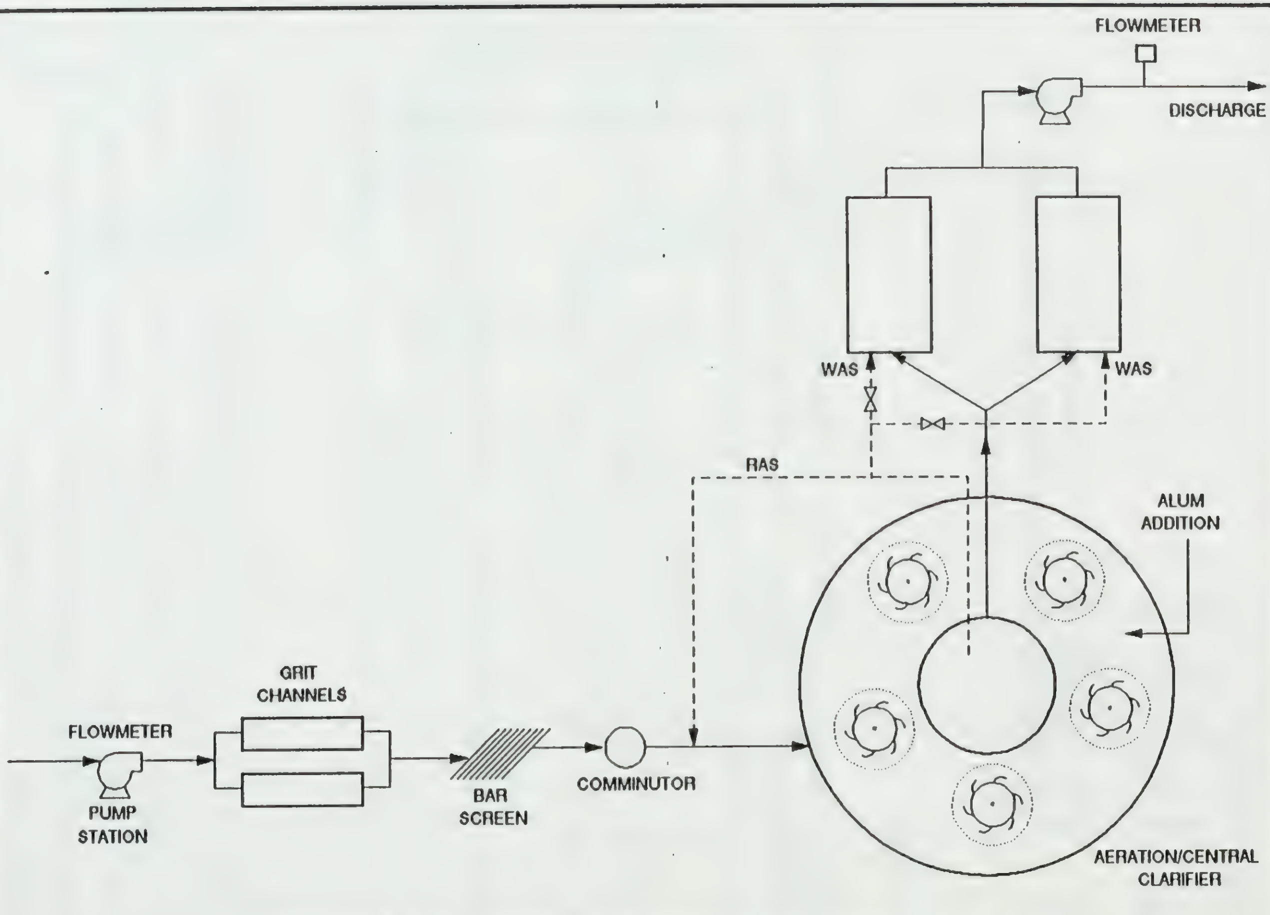


FIGURE A1.22 - SCHEMATIC LAYOUT OF COOKSTOWN WPCP

TABLE A1.13 DESIGN CRITERIA FOR COOKSTOWN	
DESIGN CAPACITY (m^3/d)	825 2634(peak)
YEAR OF START-UP	1988
PRELIM.TREATMENT	
Screening	Coarse
Grit Removal	Yes
Comminutor	Yes
AERATION	
Design HRT (hrs)	28
Operating SRT	
summer (d)	NA
winter (d)	NA
Desgin Organic Loading	
($g/1000\ m^3/d$)	145.5
AERATION METHOD	AIRE-02
kW	12.5
kW/1000 m^3 tankage	13.0
CLARIFICATION	
Number	One
SOR @ Design Flow (m^3/m^2d)	10.5
SOR @ Peak Flow (m^3/m^2d)	33.6
RAS Ratio (% of flow)	50 to 200
Surface Skimmer	No
PHOSPHORUS REMOVAL	
Chemical	Alum
Addition Points	aeration effluent
LAGOONS	
No. of Cells	Two
Surface Area ($ha/1000\ m^3/d$)	6.0
HRT @ Design Flow (d)	122
SLUDGE HOLDING TANK	No

N/A - Not Applicable

TABLE A1.14 EFFLUENT LIMITS FOR COOKSTOWN WPCP			
Parameter	Basis	Concentration (mg/L)	Loading
BOD_5	Annual Average	25.	NA
TSS	Annual Average	25.	NA
TP	Monthly Average	1.0	300 kg/y
Total NH_3-N	Annual Average	4.0	NA
	Monthly Average (Apr/Oct/Nov)	4.0	NA
Discharge	April	NA	60 L/S
	Oct to Mar	NA	10 L/S
	May	NA	10 L/S
	June 1 - Sept 30	NA	No Discharge

NA = Not Applicable

Operation and Performance

Raw sewage, clarifier and lagoon effluents are sampled every two weeks and submitted to the MOE for analysis. Raw sewage is a 24 hour composite sample and all others are grab samples. Extensive in-plant monitoring and analysis is conducted for process control.

Table A1.15 summarizes the operating conditions and performance of the Cookstown WPCP for 1989 and 1990. The design HRT for the aeration tank is 28 hours. The operating HRT was 93 and 59 hours in 1989 and 1990, respectively. The design organic loading at Cookstown is 146 kg/1000 m^3.d. Actual organic loading for 1989 and 1990 was 38 and 79 kg/1000 m^3.d, respectively. There is relatively little performance data on the plant. Final effluent is only sampled when the lagoon is being discharged and discharges have been limited to high flow periods in April.

TABLE: A1.15 **Current Operating Conditions**

Cookstown WPCP

YEAR	1989	1990
AVERAGE DAY FLOW (m3)	248	389
MAXIMUM DAY FLOW (m3)	613	719
RAW SEWAGE:		
AVG. BOD INF. (mg/L)	148.5	194.4
AVG. TSS INF. (mg/L)	227.1	229.2
AVG. TKN INF. (mg/L)	N/D	52.9
AVG. TP INF. (mg/L)	7.8	8.3
AERATION		
OPERATING HRT (hrs)	93.0	59.4
OPERATING SRT (d)	148	72
F/M	0.01	0.03
ORGANIC LOADING (kg/1000m3(AERAT.)*d)	38	79
CLARIFICATION		
OPERATING HRT (hrs)	25.0	16.0
SOR @ AVERAGE FLOW (m^3/m^2 d)	3.5	5.5
SOR @ PEAK DAY FLOW (m^3/m^2 d)	8.5	10.0
SECONDARY EFFLUENT:		
AVG. BOD (mg/L)	4.6	7.9
STD.DEV.	1.6	3.5
AVG. TSS (mg/L)	14.9	15.5
STD.DEV.	6.5	9.1
AVG. TKN (mg/L)	1.2	1.2
STD.DEV.	0.5	0.4
AVG. NH3-N (mg/L)	0.2	0.1
STD.DEV.	0.4	0.0
AVG. NO(T)-N (mg/L)	23.0	18.7
STD.DEV.	7.8	2.3
AVG. TP (mg/L)	0.9	0.5
STD.DEV.	0.2	0.4
LAGOON (receiving sludge)		
OPERATING HRT (d)	405.4	258.9
LAGOON EFFLUENT:		
AVG. BOD (mg/L)	2.3	8.0
STD.DEV.	1.9	4.6
AVG. TSS (mg/L)	5.0	N/D
STD.DEV.	4.0	N/D
AVG. TKN (mg/L)	N/D	2.7
STD.DEV.	N/D	1.2
AVG. NH3-N (mg/L)	N/D	0.9
STD.DEV.	N/D	1.2
AVG. NO(T)-N (mg/L)	N/D	N/D
STD.DEV.	N/D	N/D
AVG. TP (mg/L)	0.3	0.3
STD.DEV.	0.2	0.1
SLUDGE		
ESTIMATED ANNUAL SLUDGE LOADING ON LAGOON (kg/m3 of lagoon) receiving sludge	0.1	0.2
ESTIMATED SLUDGE PRODUCED (kg/d)	32.3	66.1
TOTAL ESTIMATED SLUDGE DISCHARGED TO LAGOON (tonnes) (since start-up)	72	

The clarifier SORs are both below the design SOR of 10.5 $m^3/m^2.d$ based on average day flow, and peak day SORs for 1989 and 1990 were below the design value of 13.0 $m^3/m^2.d$. The design HRT for the two lagoons is 122 days. In 1989 and 1990 the HRTs were 405 and 259 days, respectively. These extremely long retention times may be attributed to an average day flow which is less than 50 percent of the average day design flow.

MLSS levels are maintained at 3000 mg/L in the summer and 4000 mg/L in the winter. MLSS is sampled three times per week. A sludge volume of 55 m^3 is wasted daily five days a week with more sludge being wasted to lagoon #1 than to lagoon #2 due to the valve arrangement. Grit is removed from the grit channels approximately every two weeks and discharged to the lagoon.

The original aerators at the Cookstown WPCP have required frequent removal for maintenance. To provide oxygen transfer capability when these aerators are out-of-service, two submerged aspirating aerators have been installed in the aeration basin as standby units. In addition, final effluent is pumped continuously as flushing water to the shafts of the Aire-O_2 units.

All flow to the treatment plant is either commercial or residential.

Sludge Loading and Sludge Accumulation

Based on operating conditions for 1989 and 1990, the annual sludge loading on the Cookstown lagoons was 0.1 and 0.2 kg/m^3 of lagoon, respectively. The estimated sludge produced for 1989 and 1990 was 32.3 and 66.1 kg/d, respectively. Using the average of the sludge produced for 1989 and 1990, approximately 72 tonnes of sludge has been discharged to the lagoon since start-up. No sludge mapping was done at the Cookstown WPCP.

A1.5 LINDSAY WPCP

Background

The Lindsay WPCP was modified to the Sutton Process and commissioned in January of 1990. Prior to this, the plant consisted of two parallel trains of aerated lagoons, followed by three facultative ponds. These lagoons had existed since 1961, with minor modifications to influent location in 1976. The Sutton Process modifications included conversion of the north aerated cell to an aeration basin and conversion of the south aerated cell to a clarifier and sludge holding tank. New alum addition points were added to the aeration basin and clarifier effluents. The influent points to the lagoons were changed at this time, but no sludge was removed from the facultative ponds. Design capacity of this Sutton Process plant is 15,870 m^3/d.

Process Description

A flow schematic of the Lindsay WPCP is shown in Figure A1.23. Raw sewage passes through a coarse screen at the headworks of the treatment plant. The north aeration cell (earthen basin) was expanded to provide 22,750 m^3 of aeration capacity. Air is supplied by eight 18.6 kW AIRE-02 aerators, two 11 kW and two 15 kW WELLS surface aerators. Alum is added to the aeration effluent prior to clarification.

The clarifier has a diameter of 34.5 m and is approximately 4 m in depth. There are separate RAS and WAS pumps. A sludge holding lagoon was provided for excess sludge storage. WAS sludge was originally pumped to the sludge holding tank prior to discharge to the lagoons. This was discontinued in January of 1991 due to odour problems. A Parshall flume and ultrasonic sensor measures effluent from the clarifier prior to discharge to the lagoons. The south lagoon receives substantially more flow than the north lagoon. There is no level equalization between the two lagoons.

Table A1.16 and Table A1.17 summarize the design criteria and effluent limits respectively.

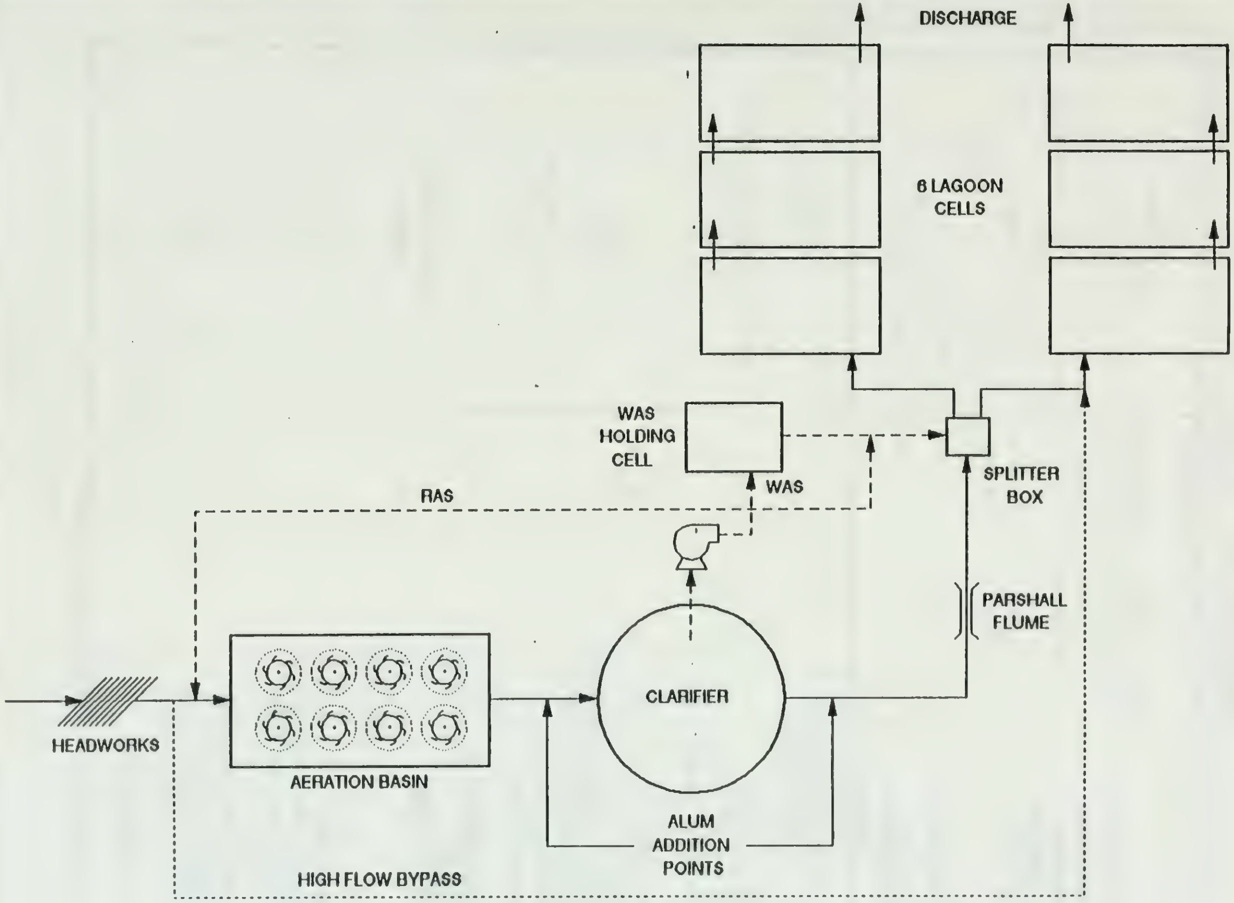


FIGURE A1.23 - SCHEMATIC LAYOUT OF LINDSAY WPCP

TABLE A1.16

DESIGN CRITERIA FOR LINDSAY WPCP

DESIGN CAPACITY (m^3/d)	15,870 22,220(peak)
YEAR OF START-UP	1990
PRELIM.TREATMENT	Coarse
Screening	No
Grit Removal	No
Comminutor	
AERATION	
Design HRT (hrs)	34 24.0 (peak)
Operating SRT	30 to 40
summer (d)	
winter (d)	
Design Organic Loading (g/1000 m^3/d)	117.9
AERATION METHOD	AIRE-02 and high speed WELLS
kW	148.8 and 52.2
kW/1000 m^3 tankage	9.0
CLARIFICATION	
Number	One
SOR @ Design Flow (m^3/m^2d)	17.0
SOR @ Peak Flow (m^3/m^2d)	24.3
RAS Range (% of flow)	Up to 100% peak
Surface Skimmer	No
PHOSPHORUS REMOVAL	
Chemical	Alum
Addition Points	aeration & clarifier effluents
LAGOONS	
No. of Cells	Six
Surface Area (ha/1000 m^3/d)	3.0
HRT @ Design Flow (d)	54
LAGOON RECEIVING SLUDGE	
Surface Area (ha)	16.0
HRT @ Design Flow (d)	18
SLUDGE HOLDING TANK	Yes

TABLE A1.17

EFFLUENT LIMITS FOR LINDSAY WPCP

Parameter	Basis	Concentration	Loading (kg/y)
BOD_5	Annual Average	15.0	NA
TSS	Annual Average	15.0	NA
TP	Monthly Average	0.30	1400

NA = Not Applicable

The Lindsay WPCP has annual limits for BOD, TSS, and phosphorus outlined in the Certificate of Approval. The Certificate of Approval also limits plant flow to 12,672 m^3/d.

Operation and Performance

Grab samples of raw sewage, aeration, clarifier and lagoon effluents are sampled weekly and submitted to the MOE for analysis. Measurements of DO, OUR, MLSS, RAS/WAS and 30 minute settling are taken daily. The plant also conducts regular microscopic exams, ammonia and phosphorus sampling using a Hach kit.

Table A1.18 summarizes the operating conditions at the Lindsay WPCP for 1991. The design HRT for the aeration tank is 34 hours, and the operating HRT was 42.4 hours. The operating SRT was 93 days compared to the design value of 30 to 40 days. The design organic loading at Lindsay is 118 kg/1000 m^3.d. Actual organic loading for 1991 was 45 kg/1000 m^3.d.

TABLE: A1.18 **Current Operating Conditions**

Lindsay WPCP

YEAR	1991
AVERAGE DAY FLOW (m3)	12577
MAXIMUM DAY FLOW (m3)	37160
RAW SEWAGE:	
AVG. BOD INF. (mg/L)	79.9
AVG. TSS INF. (mg/L)	99.9
AVG. TKN INF. (mg/L)	24.4
AVG. TP INF. (mg/L)	3.6
AERATION	
OPERATING HRT (hrs)	42.4
OPERATING SRT (d)	93
F/M	0.02
ORGANIC LOADING (kg/1000m3(AERAT.)*d)	45
CLARIFICATION	
OPERATING HRT (hrs)	7.1
SOR @ AVERAGE FLOW (m^3/m^2 d)	13.5
SOR @ PEAK DAY FLOW (m^3/m^2 d)	39.7
SECONDARY EFFLUENT:	
AVG. BOD (mg/L)	* 5.0
STD.DEV.	3.6
AVG. TSS (mg/L)	7.3
STD.DEV.	4.1
AVG. TKN (mg/L)	1.1
STD.DEV.	0.4
AVG. NH3-N (mg/L)	0.2
STD.DEV.	0.3
AVG. NO(T)-N (mg/L)	10.3
STD.DEV.	4.5
AVG. TP (mg/L)	0.6
STD.DEV.	0.3
LAGOON (receiving sludge)	
OPERATING HRT (d)	23
LAGOON EFFLUENT:	
AVG. BOD (mg/L)	3.3
STD.DEV.	2.5
AVG. TSS (mg/L)	7.2
STD.DEV.	6.0
AVG. TKN (mg/L)	1.7
STD.DEV.	0.6
AVG. NH3-N (mg/L)	0.5
STD.DEV.	0.5
AVG. NO(T)-N (mg/L)	6.3
STD.DEV.	3.4
AVG. TP (mg/L)	0.2
STD.DEV.	0.1
SLUDGE	
ESTIMATED ANNUAL SLUDGE LOADING ON LAGOON (kg/m3 of lagoon) receiving sludge	1.1
ESTIMATED SLUDGE PRODUCED (kg/d)	824.0
TOTAL ESTIMATED SLUDGE DISCHARGED TO LAGOON (tonnes) (since start-up)	8

* Linsday lagoon effluent average of north and south outfall

The clarifier SOR was below the design SOR of 17 m^3/m^2d based on average day flow, and above the design value of 24.3 m^3/m^2d for peak day SOR. The design HRT for the two lagoons is 54 days, and the 1991 operating HRT was 23 days.

The treatment facility was initially operated with a MLSS of 4000 to 5000 mg/L, but the staff had difficulty maintaining a DO concentration above zero. Currently the plant operates at a MLSS of 2400 to 2600 mg/L in the summer, and an optimum winter level is still being determined. Sludge is wasted about 3 times per day for 1 to 2 hour timed cycles. A sludge holding tank was used to store sludge prior to discharge, but was discontinued in January 1991 due to floating sludge and odour problems. Sludge is now wasted directly to the lagoons.

Most of the flow to the facility is either commercial or residential except for the leachate from two landfills. Some septic and holding tank wastes are brought into the facility.

Sludge Loading and Sludge Accumulation

Based on operating conditions for 1991, the annual sludge loading on the Linsday WPCP lagoons was 1.1 kg/m^3 of lagoons. The estimated sludge produced for 1991 was 824 kg/d. Using this value, approximately eight tonnes of sludge have been discharged to the lagoons since start-up.

A1.6 DUTTON WPCP

Background

The Sutton Process was commissioned at the Dutton WPCP in 1990. The original treatment consisted of a single facultative lagoon. In addition to adding an extended aeration plant, the inlet and outlet of the lagoon was modified. The rated capacity of the new facility is 558 m^3/d. No sludge was removed from the lagoon at the time of upgrading.

Process Description

A flow schematic of the Dutton WPCP is shown in Figure A1.24. Raw sewage enters the treatment plant and passes through a screening/flow spitting chamber. The bar screen is manually cleaned. Flow is split between two aeration basins each measuring 20.7 by 6.6 m with a depth of 3.0 m. Air is supplied by three 7.6 kW mechanical surface aerators per aeration basin. Alum can be added to the clarifier influent or the clarifier effluent. Lime can also be added to the aeration basin as a source of supplemental alkalinity.

The single clarifier has a diameter of 9.75 m and a depth of 3.66 m. RAS is pumped back to the influent of the aeration basins and flow is metered using a magnetic flow meter. A scum scraper was removed due to freezing in the winter months. WAS is wasted to the lagoon by changing the valving of the RAS pumps. WAS, scum and clarified effluent are discharge to the lagoon at the same point. There is an ultrasonic sensor and V-notch weir on the effluent of the clarifier for flow measurement. The lagoon has a surface area of approximately 4.0 ha.

Table A1.19 and Table A1.20 summarize the design criteria and effluent limits respectively. The Dutton WPCP has daily and monthly limits for BOD, TSS, ammonia and phosphorus outlined in the Certificate of Approval.

Operation and Performance

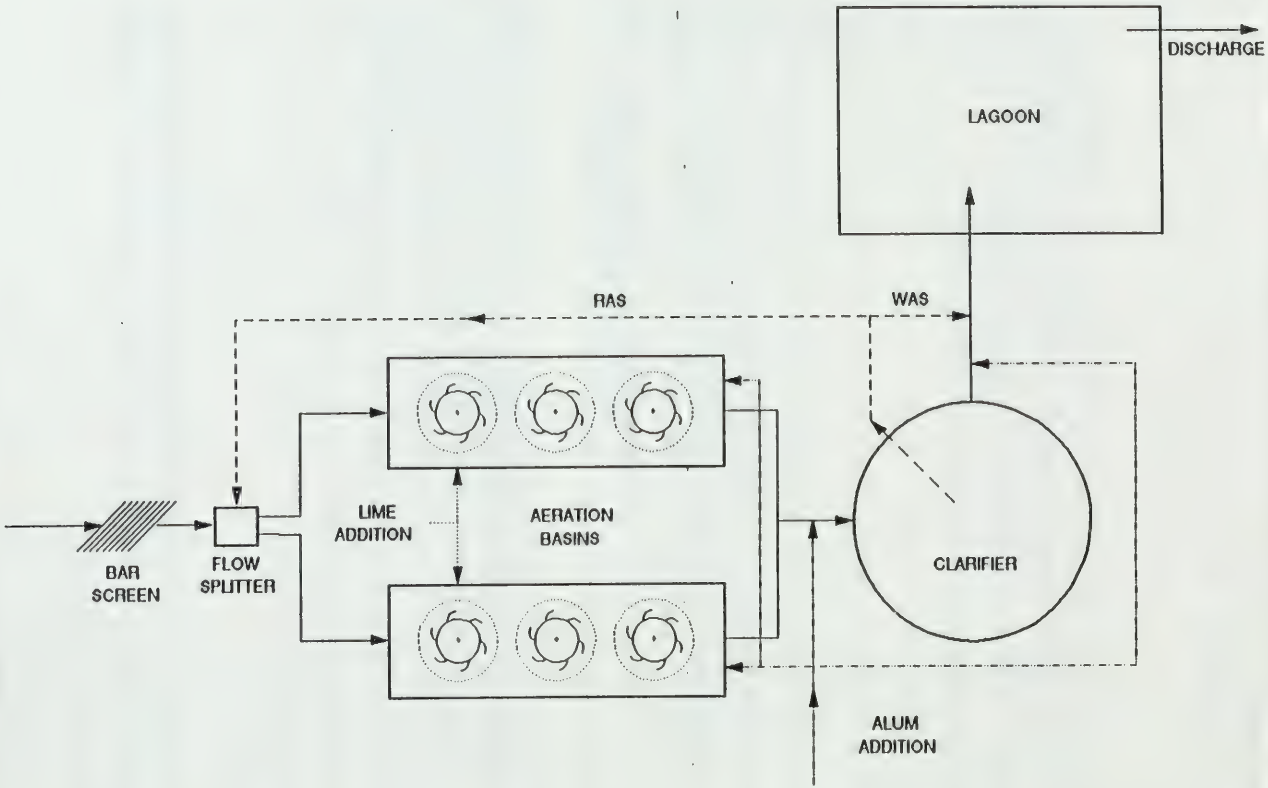


FIGURE A1.24 - SCHEMATIC LAYOUT OF DUTTON WPCP

TABLE A1.19

DESIGN CRITERIA FOR DUTTON WPCP

DESIGN CAPACITY (m^3/d)	558 1,885(peak)
YEAR OF START-UP	1991
PRELIM.TREATMENT	
Screening	Coarse
Grit Removal	No
Comminutor	No
AERATION	
Design HRT (hrs)	35
Operating SRT	NA
summer (d)	
winter (d)	
Design Organic Loading (kg/1000m^3*d)	113.4**
AERATION METHOD	Low Speed Surface Aeration
kW	45.6
- kW/1000 m^3 tankage	56.5
CLARIFICATION	
Number	One
SOR @ Design Flow (m^3/m^2d)	7.5
SOR @ Peak Flow (m^3/m^2d)	25.2
RAS Ratio (% of flow)	Up to 200%
Surface Skimmer	removed
PHOSPHORUS REMOVAL	
Chemical	Alum
Addition Points	aeration & clarifier effluents
LAGOONS	
No. of Cells	One
Surface Area (ha/1000 m^3/d)	7.3
HRT @ Design Flow (d)	86
SLUDGE HOLDING TANK	No

** Based on 1989[BODinf]

N/A - Not Applicable

TABLE A1.20

EFFLUENT LIMITS FOR DUTTON WPCP

Parameter	Basis	Concentration (mg/L)	Loading (kg/d)
BOD	Monthly Average, Winter	15.	6.51
	Monthly Average, Summer	10.	
	Daily Maximum, Winter	25.	
	Daily Maximum, Summer	15.	
TSS	Monthly Average, Winter	15.	6.51
	Monthly Average, Summer	10.	
	Daily Maximum, Winter	25.	
	Daily Maximum, Summer	15.	
TP	Monthly Average, Winter	1.0	0.37
	Monthly Average, Summer	0.5	
	Daily Maximum, Winter	1.5	
	Daily Maximum, Summer	1.0	
Total NH_3-N	Monthly Average, Winter	5.00	2.05
	Monthly Average, Summer	3.00	
	Daily Maximum, Winter	7.50	
	Daily Maximum, Summer	4.50	

The Certificate of Approval requires a minimum of bi-weekly sampling of raw sewage, clarifier and lagoon effluents. Clarifier effluent can be a grab sample, but raw sewage and lagoon effluent must be 24 hour composites.

The treatment plant was in the process of stating up in the fall of 1991. Hence, no performance data are available.

A1.7 RODNEY WPCP

Background

The Sutton Process was commissioned in the fall of 1991. The original treatment plant consisted of a single facultative lagoon operated as seasonal discharge with batch treatment with alum. In addition to adding an extended aeration plant, the outlet of the lagoon was modified. The rated capacity of the new facility is 590 m^3/d. No sludge was removed from the lagoon at the time of upgrading.

Process Description

A flow schematic of the Rodney WPCP is shown in Figure A1.25. Raw sewage enters the treatment plant and passes through a screening/flow spitting chamber. The bar screen is manually cleaned. Flow is split between two aeration basins each measuring 20.7 by 6.6 m with a depth of 3.0 m. Air is supplied by three 7.6 kW mechanical surface aerators per aeration basin. Alum can be added to the clarifier influent or the clarifier effluent. Lime can also be added to the aeration basin as a source of supplemental alkalinity.

The clarifier has a diameter of 9.75 m and a depth of 3.66 m. A surface skimmer collects scum and discharges it directly to the lagoon. RAS is pumped back to the influent of the aeration basins and flow is metered using a magnetic flow meter. WAS is wasted by changing the valving of the RAS pumps. RAS is discharged to the lagoon at the same point as clarifier scum and clarified effluent. There is an ultrasonic sensor and V-notch weir on the effluent of the clarifier for flow measurement. The lagoon has a surface area of approximately 6.5 ha.

Table A1.21 and Table A1.22 summarize the design criteria and effluent limits respectively. The Rodney WPCP has daily and monthly limits for BOD, TSS, ammonia and phosphorus outlined in the Certificate of Approval.

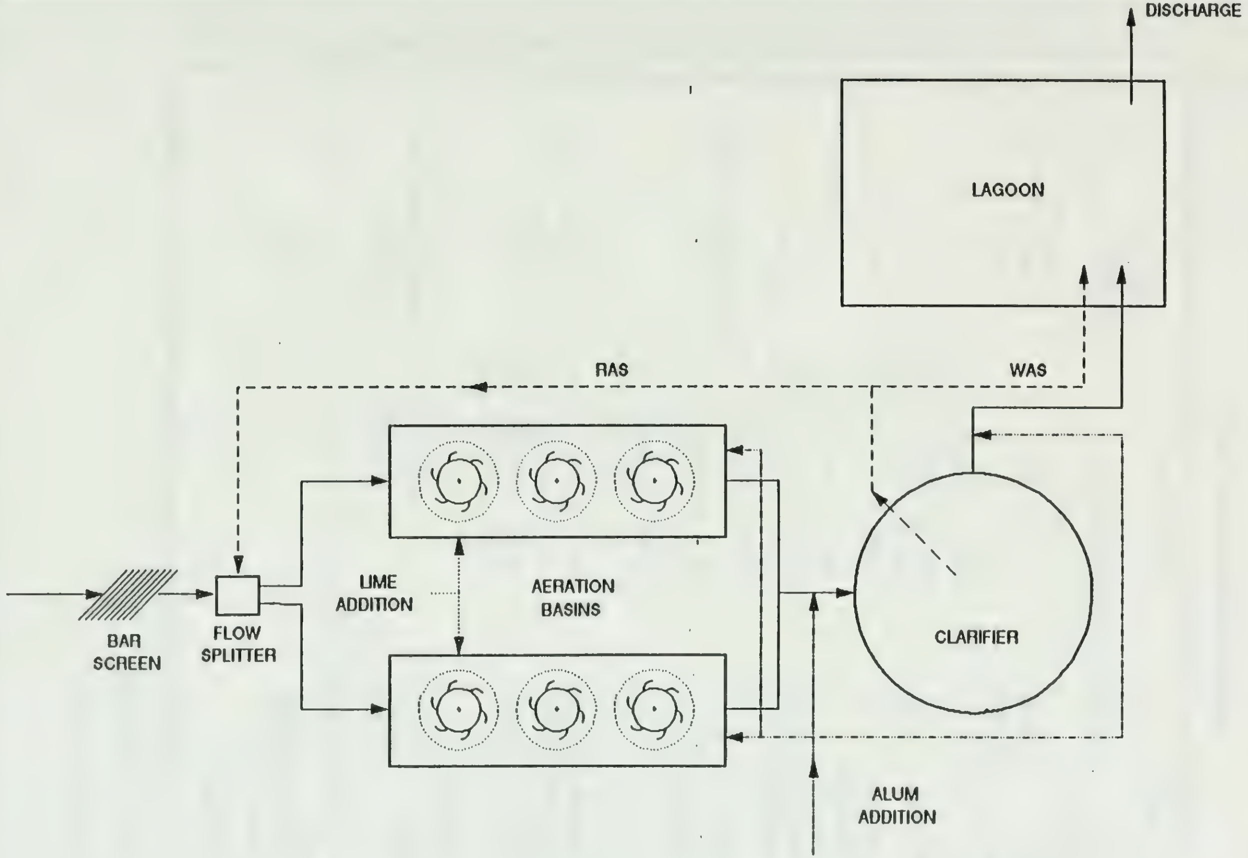


FIGURE A1.25 - SCHEMATIC LAYOUT OF RODNEY WPCP

TABLE A1.21 DESIGN CRITERIA FOR RODNEY	
DESIGN CAPACITY (m^3/d)	590 2,190(peak)
YEAR OF START-UP	1991
PRELIM.TREATMENT	
Screening	Coarse
Grit Removal	No
Comminutor	No
AERATION	
Design HRT (hrs)	33
Operating SRT	NA
summer (d)	
winter (d)	
Design Organic Loading ($kg/1000m^3*d$)	200.0**
AERATION METHOD	Low Speed Surface Aeration
kW	44.7
- kW/1000 m^3 tankage	55.1
CLARIFICATION	
Number	One
SOR @ Design Flow (m^3/m^2d)	7.9
SOR @ Peak Flow (m^3/m^2d)	29.3
RAS Ratio (% of flow)	Up to 200%
Surface Skimmer	Yes
PHOSPHORUS REMOVAL	
Chemical	Alum
Addition Points	aeration & clairfier effluents
LAGOONS	
No. of Cells	One
Surface Area (ha/1000 m^3/d)	11.0
HRT @ Design Flow (d)	40
SLUDGE HOLDING TANK	No

** Based on 1989[BODinf]

N/A - Not Applicable

TABLE A1.22

EFFLUENT LIMITS FOR RODNEY WPCP

Parameter	Basis	Concentration (mg/L)	Loading (kg/d)
BOD	Monthly Average, Winter	15.	3.90
	Monthly Average, Summer	10.	
	Daily Maximum, Winter	25.	
	Daily Maximum, Summer	15.	
TSS	Monthly Average, Winter	15.	3.90
	Monthly Average, Summer	10.	
	Daily Maximum, Winter	25.	
	Daily Maximum, Summer	15.	
TP	Monthly Average, Winter	1.0	0.28
	Monthly Average, Summer	0.5	
	Daily Maximum, Winter	1.5	
	Daily Maximum, Summer	1.0	
Total NH_3-N	Monthly Average, Winter	5.00	1.57
	Monthly Average, Summer	3.00	
	Daily Maximum, Winter	7.50	
	Daily Maximum, Summer	4.50	
Total Residual Chlorine	Monthly Average	0.01	NA
	Daily Maximum	0.03	

Operation and Performance

The Certificate of Approval requires a minimum of bi-weekly sampling of raw sewage, clarifier and lagoon effluents. Clarifier effluent can be a grab sample, and raw sewage and lagoon effluent must be 24 hour composites.

The treatment plant was in the process of stating up in the fall of 1991. Hence, no performance data are available.

A1.8 STAYNER WPCP

Background

The Sutton Process was implemented at Stayner in March 1991. The original treatment plant consisted of an aerated cell and two facultative ponds. The extended aeration plant was added to provide a design capacity of 1,875 m^3/d. The aerated cell was decommissioned and filled. The two original facultative ponds were deepened by 0.3 m to a total depth of 2.1 m. Two additional ponds are proposed for the facility to provide effluent storage.

Process Description

A flow schematic of the Stayner WPCP is shown in Figure A1.26. Raw sewage enters the plant pumping station and passes through a coarse screen. The influent flow is metered at the pumping station using a timer and magmeter. There are no grit removal facilities. Screened flow is split between two aeration tanks. Each tank is 24.5 by 24.5 m with a depth of 3.6 m. Air is provided by 44.7 kW surface aerators with draft tubes. Alum can be added to either the aeration influent or effluent.

There are two clarifiers, each having a diameter of 16 m. RAS is pumped to the aeration influent by two variable speed pumps but the volume is not metered. WAS is discharged to the lagoon from a RAS splitter box separate from the clarified effluent. The splitter box allows discharge of extended aeration plant effluent to either lagoon #1 or #2. Sludge can only be discharged to lagoon #2.

Effluent from the lagoons is automatically flow paced to the flow in the receiving stream. There is no summer discharge allowed.

Table A1.23 and Table A1.24 summarize the design criteria and effluent limits respectively. The Stayner WPCP has annual limits for BOD, TSS, and phosphorus outlined in the

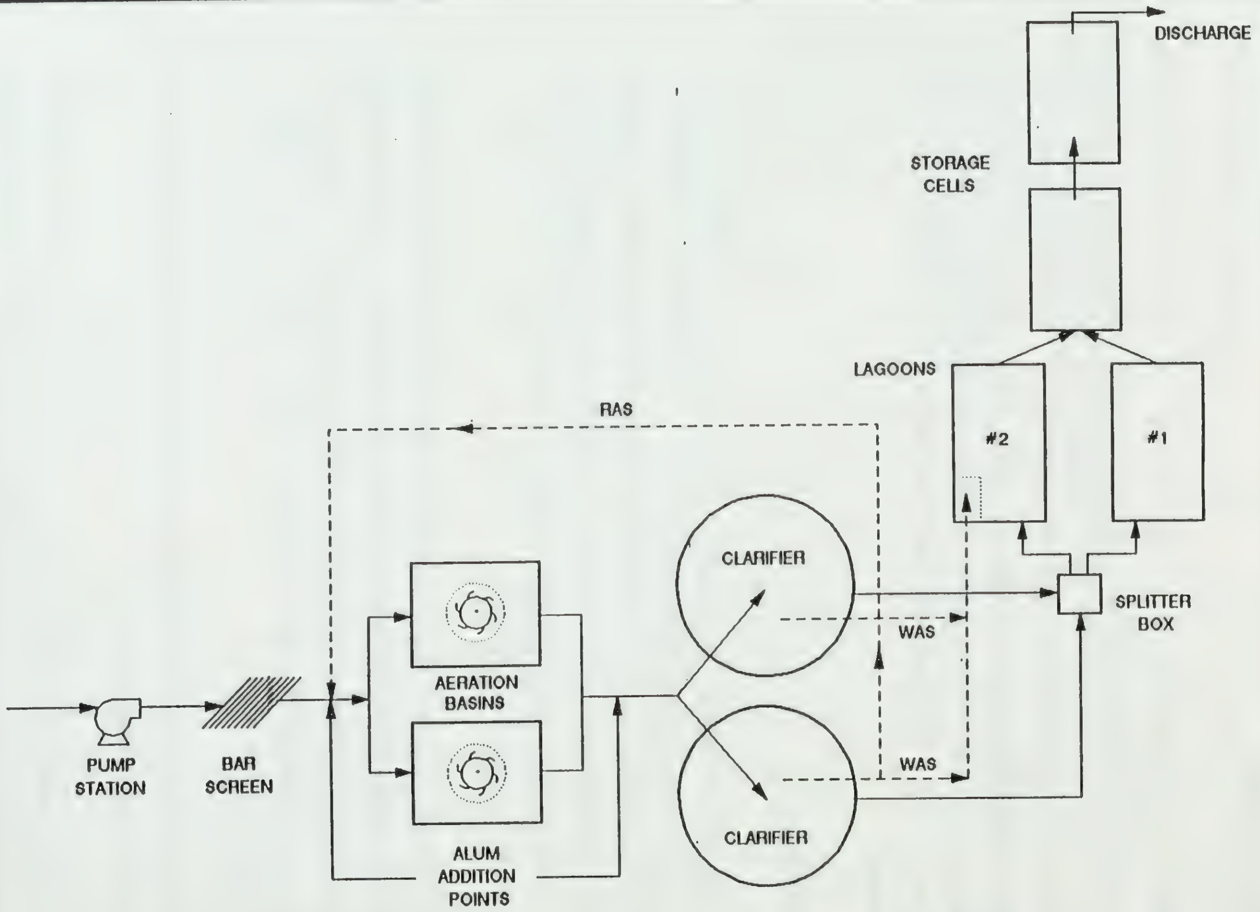


FIGURE A1.26 - SCHEMATIC LAYOUT OF STAYNER WPCP

TABLE A1.23

DESIGN CRITERIA FOR STAYNER WPCP

DESIGN CAPACITY (m^3/d)	1,875
YEAR OF START-UP	1991
PRELIM.TREATMENT	
Screening	Coarse
Grit Removal	No
Comminutor	No
AERATION	
Design HRT (hrs)	55
Operating SRT	NA
summer (d)	
winter (d)	
Design Organic Loading	
($kg/1000m^3*d$)	255.7**
AERATION METHOD	Low Speed Surface Aeration
kW	44.7
kW/1000 m^3 tankage	
CLARIFICATION	
Number	Two
SOR @ Design Flow (m^3/m^2d)	4.7
SOR @ Peak Flow (m^3/m^2d)	
RAS Ratio (% of flow)	100%
Surface Skimmer	No
PHOSPHORUS REMOVAL	
Chemical	Alum
Addition Points	aeration influent or effluent
LAGOONS	
No. of Cells	Four
Surface Area (ha/1000 m^3/d)	11.6
HRT @ Design Flow (d)	192
LAGOON RECEIVING SLUDGE	
Surface Area (ha)	3.9
HRT @ Design Flow (d)	–
SLUDGE HOLDING TANK	No

** Based on 1989[BODinf]

NA = Not Available

TABLE A1.24

EFFLUENT LIMITS FOR STAYNER WPCP

Parameter	Basis	Concentration (mg/L)	Loading
BOD_5	Annual Average	15.0	NA
TSS	Annual Average	15.0	NA
TP	Two Consecutive Samples	0.5*	320 kg/y
Total NH_3-N	Apr 1 - May 15 Sept 1 - Oct 31 Nov 1 - Mar 31	2.00 2.00 4.00	NA
Flow			No summer discharge Dilution must exceed 3:1

* TP revised to 1.0 mg/L in 1990

NA = Not Available

Certificate of Approval. There are specific monthly limits for ammonia.

Operation and Performance

Raw sewage, extended aeration effluent, lagoon effluent (if discharging), MLSS and RAS are sampled every two weeks and submitted to the MOE. Raw sewage and extended aeration effluent are 24 hour composites, and lagoon effluent is an 8 hour composite sample. The plant also samples three times per week for in-house analysis.

The major industrial discharger to the treatment plant is a fruit packer, resulting in a high seasonal raw sewage BOD.

Plant operators plan to operate extended aeration facility at 3000 mg/L MLSS in the summer and 4500 mg/L MLSS in the winter. The plant was in the process of start-up in 1991. Hence, no performance data are available.

A2.1 NEW HAMBURG WPCP

Background

The New Hamburg facility was designed in 1964 as two facultative ponds which could be operated in parallel or series. Effluent quality deterioration led to pilot testing of intermittent filtration and construction to upgrade the facility began in 1980. By 1981, the facility was operating as a New Hamburg process with a design capacity of 2,700 m^3/d. The upgrading included the addition of an aerated cell prior to the existing lagoons, and a sand filter following the facultative lagoons. The east facultative cell was deepened from 2.3 m to 3.0 m. Following 16 years of operation, approximately 8 cm of sludge was removed from the lagoons. Final effluent is discharged to a constructed wetland.

Process Description

A flow schematic of the New Hamburg WPCP is shown in Figure A2.1. Raw sewage is pumped to the treatment plant and enters the aerated lagoon. The aerated lagoon is lined and has a volume of 11 600 m^3. Aeration is supplied by twenty-four 200 mm non-clog Mat aerators and twenty-eight 300 mm regular Mat aerators. Air is supplied to four headers from one of three 18.6 kW compressors.

Alum is added to the aeration cell effluent before it enters the first of two facultative lagoons in series. The first lagoon (east cell) is 3.0 m deep and has a surface area of 5.7 ha. The second lagoon (west cell) is 2.3 m deep and has a surface area of 5.4 ha.

Effluent from the facultative lagoon is pumped by two 15 kW pumps to the sand filters. There are four lined sand filters, each with a surface area of 0.12 ha. Distribution of flow is through a perforated pipe spray system. The underdrain is a perforated PVC pipe collecting filtered water which passes through a Parshall flume before final discharge to the wet land. Table A2.1 and A.2 summarize the design criteria and effluent quality parameters respectively for the New Hamburg WPCP.

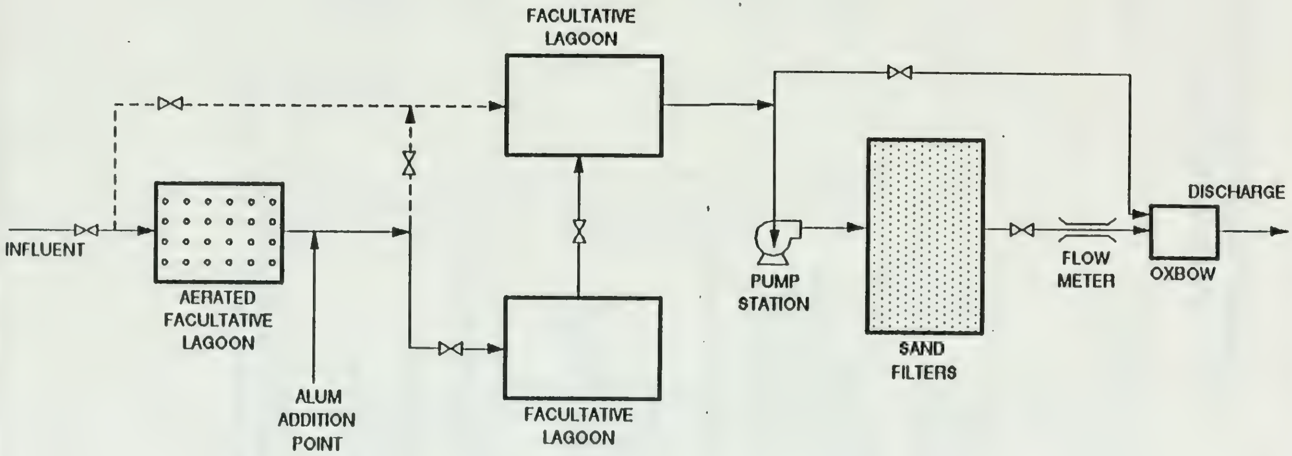


FIGURE A2.1 - SCHEMATIC LAYOUT OF NEW HAMBURG WPCP

TABLE: A2.1

SUMMARY OF DESIGN CRITERIA FOR NEW HAMBURG WPCP

DESIGN CAPACITY (m^3/d)	2 700**
YEAR OF START-UP	1981
PRELIM. TREATMENT	
SCREENING	NO
GRIT REMOVAL	NO
AERATED CELL	
DESIGN HRT (d)	4.2
BOD LOADING (kg/m^3d)	0.05
(kg/m^2d)	0.10
AERATION PROCESS	3 @ 18.6 kW blowers 52 MATT aerators
AIR FLOW (L/min per m^3)	2.20
FACULTATIVE CELLS	
NUMBER	TWO
SURFACE AREA (ha)	11.1
RETENTION TIME (d)	102
FILTER	
NUMBER	FOUR
SURFACE LOADING (L/m^2 d) *	562.5
PHOSPHORUS REMOVAL	
CHEMICAL	ALUM
ADDITION POINT	effluent of aerated cell

* Based on design flow/total surface area

** Certificate of Approval limits flow to 2,300 m^3/d.

TABLE A2.2

EFFLUENT QUALITY PARAMETERS
FOR NEW HAMBURG WPCP

PARAMETER	BASIS		CONCENTRATION (mg/l)
BOD (mg/L)	monthly	May-Oct	15.0
		Nov-Apr	30.0
TSS (mg/L)	annual		25.0
TP (mg/L)	annual		1.0
TKN (mg/L)	Jan	@ 5 Cels.	20.0
	Feb	@ 5 Cels.	25.0
	Mar	@ 10 Cels.	25.0
	Apr	@ 15 Cels.	20.0
	May	@ 20 Cels.	10.0
	Jun	@ 25 Cels.	10.0
	Jul	@ 25 Cels.	10.0
	Aug	@ 25 Cels.	10.0
	Sep	@ 20 Cels.	10.0
	Oct	@ 15 Cels.	10.0
	Nov	@ 10 Cels.	15.0
	Dec	@ 5 Cels.	15.0
Total NH3-N (mg/L)	Jan	@ 5 Cels.	15.0
	Feb	@ 5 Cels.	20.0
	Mar	@ 10 Cels.	20.0
	Apr	@ 15 Cels.	15.0
	May	@ 20 Cels.	5.0
	Jun	@ 25 Cels.	5.0
	Jul	@ 25 Cels.	5.0
	Aug	@ 25 Cels.	5.0
	Sep	@ 20 Cels.	5.0
	Oct	@ 15 Cels.	5.0
	Nov	@ 10 Cels.	10.0
	Dec	@ 5 Cels.	10.0

The New Hamburg WPCP has monthly and annual limits for TSS and phosphorus which are based on policy 08-01 and 08-04. The Certificate of Approval limits monthly discharging of BOD, TKN and ammonia, as well as limiting the design capacity to 2,300 m^3/d.

Operation and Performance

Raw sewage, aeration cell effluent, east cell contents, west cell contents and sand filter effluent are sampled every two weeks and submitted to the MOE. The raw sewage is a four-hour composite and all others are grab samples.

Table A2.3 summarizes the facultative lagoon and sand filter effluents on an annual average basis for 1990 and 1991. Figures A2.2 through A2.6 graphically compare lagoon and filter effluent on a monthly basis based on the data from Table A2.3.

Industrial dischargers to the treatment plant include a cheese factory.

The filters are started early in March after the thaw and operated until mid-December. Two filters are operational at one time. The filters are loaded for two hours per day for approximately 24 days before they must be switched to the standby mode to dry out.

TABLE: A2.3

PERFORMANCE OF NEW HAMBURG WPCP

(1990–1991)

FACULTATIVE CELL VERSUS FILTER EFFLUENT

	BOD		SS		TP		NH3		TKN	
	CELL 2 mg/L	FILTER mg/L	CELL 2 mg/L	FILTER mg/L	CELL 2 mg/L	FILTER mg/L	CELL 2 mg/L	FILTER mg/L	CELL 2 mg/L	FILTER mg/L
1990										
AVERAGE	12.50	2.20	15.80	1.70	1.20	0.50	15.50	1.20	19.00	2.00
STD DEV	6.30	2.80	9.00	1.30	0.50	0.10	4.40	1.90	4.00	2.30
1991 *										
AVERAGE	11.50	1.80	18.00	1.10	0.70	0.40	14.30	0.60	17.60	1.10
STD DEV	2.00	0.80	9.50	0.90	0.20	0.10	4.50	0.70	3.90	0.70

* Jan.–Aug.

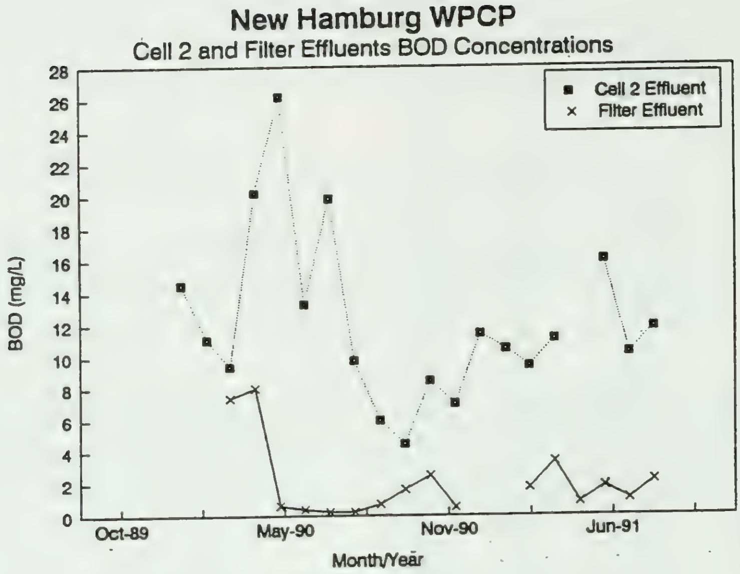


Figure A2.2

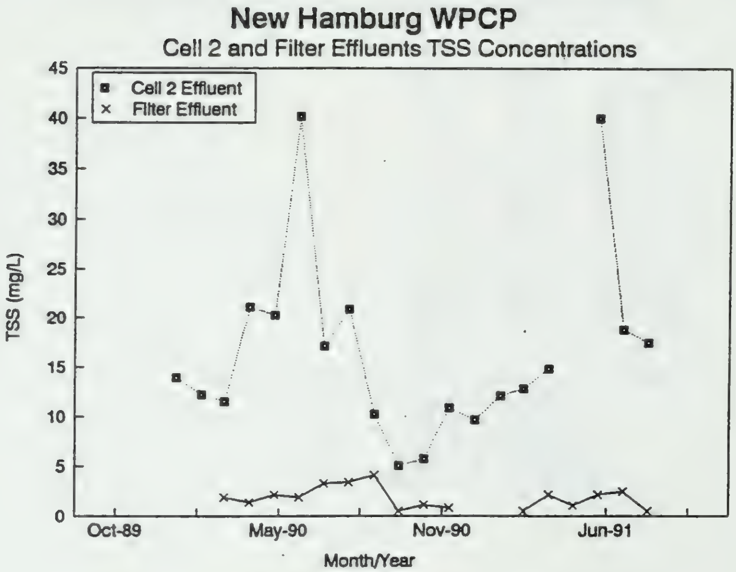


Figure A2.3

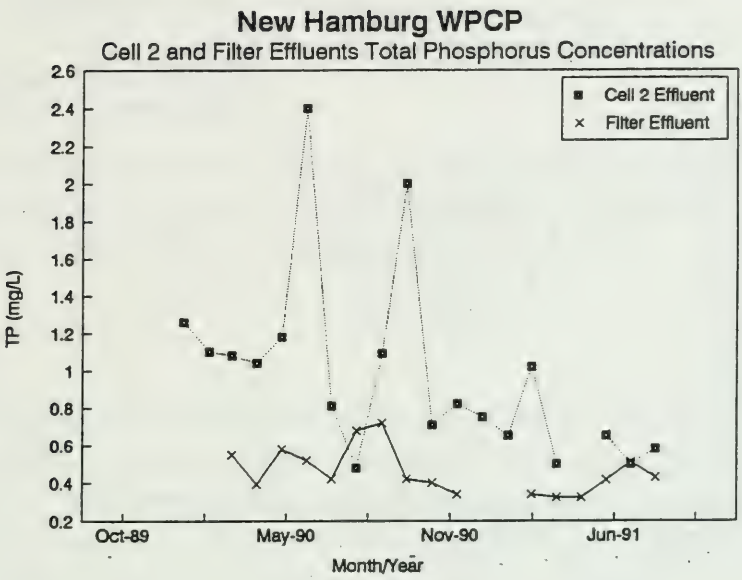


Figure A2.4

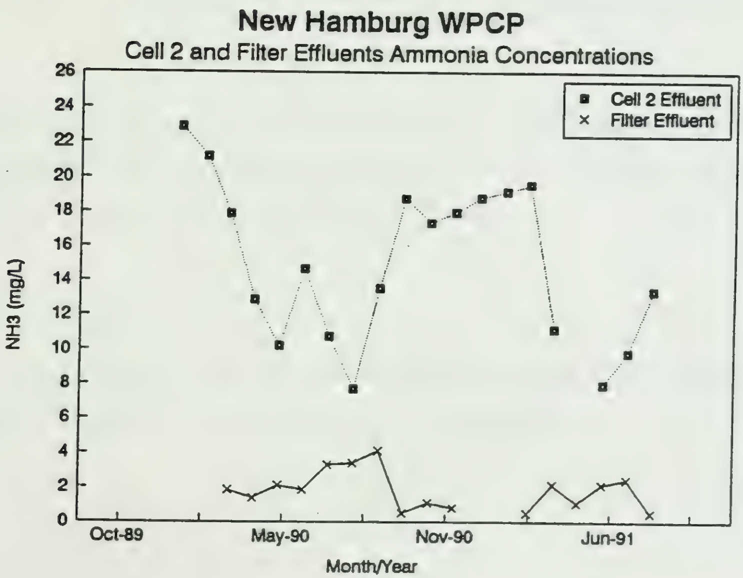


Figure A2.5

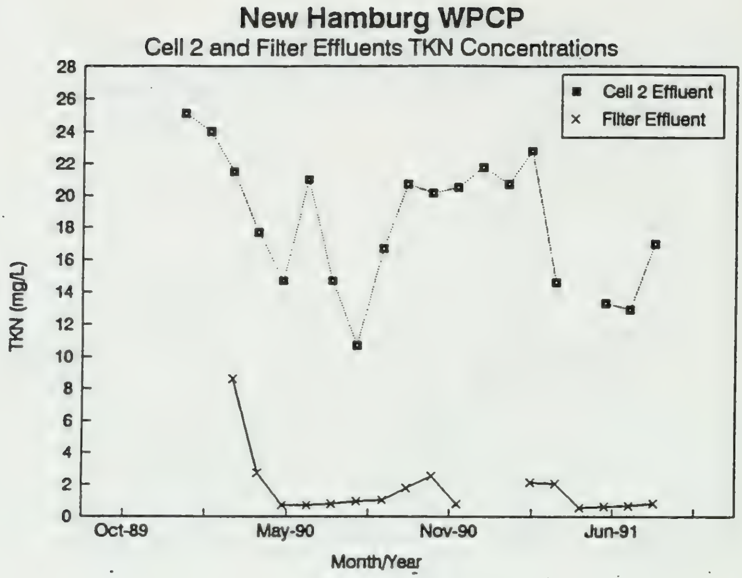

New Hamburg WPCP
Cell 2 and Filter Effluents TKN Concentrations
TKN (mg/L)
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
Cell 2 Effluent
Filter Effluent
Oct-89
May-90
Nov-90
Jun-91
Month/Year

Figure A2.6

A2.2 SCHOMBERG WPCP

Background

The Schomberg WPCP was commissioned in the fall of 1990 as a new facility. The three lagoons were filled by the fall of 1991, mostly with rainwater. Up until this time, only one of the three filter cells had been used (spring 1991).

Process Description

A flow schematic of the Schomberg WPCP is shown in Figure A2.7. Raw sewage enters the treatment plant pumping station and is measured by a volume meter in the wet well. Flow from the wet well is discharged to each of the three lagoons which are operated on a batch fill and draw basis. Each lagoon has a surface area of 2.2 ha. Alum is added to the influent to the facultative lagoons, and can also be added to the filter influent.

The three filter cells are fed from a wet well by two PLC-controlled pumps through six feed points. Each filter has a surface area of 0.24 ha. Filter effluent is collected in a wet well and is pumped to the receiving stream. Discharge flow is measured by a meter on the pump output.

Table A2.4 and Table A2.5 summarize the design criteria and effluent limits respectively. The Schomberg WPCP has monthly and annual limits for BOD, TSS, phosphorus and ammonia which are outlined in the Certificate of Approval.

Operation and Performance

Raw sewage is sampled twice per month and submitted to the MOE. When the plant is discharging. The final effluent is also sampled and submitted.

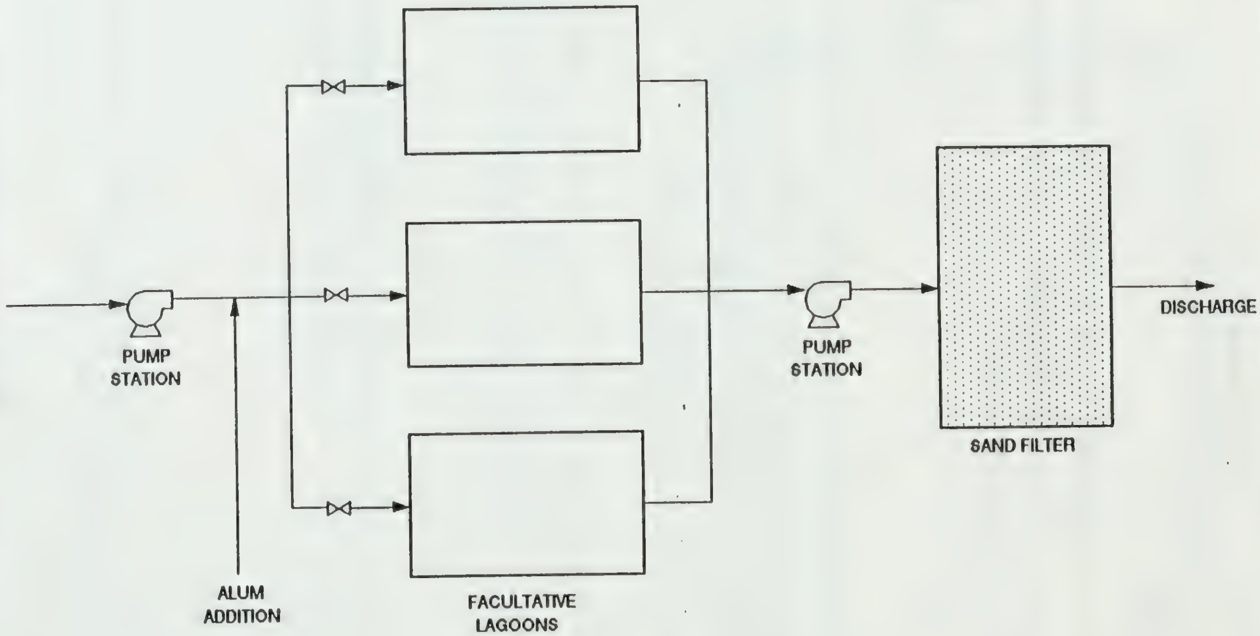


FIGURE A2.7 - SCHEMATIC LAYOUT OF SCHOMBERG WPCP

TABLE: A2.4

SUMMARY OF DESIGN CRITERIA FOR SCHOMBERG WPCP

DESIGN CAPACITY (m^3/d)	683 2595 (peak)
YEAR OF START-UP	1991
PRELIM. TREATMENT	
SCREENING	NO
GRIT REMOVAL	NO
FACULTATIVE CELLS	
NUMBER	THREE
SURFACE AREA (ha)	8.8
RETENTION TIME (d)	180
FILTER	
NUMBER	THREE
SURFACE LOADING (L/m^2 d) *	94.9
PHOSPHORUS REMOVAL	
CHEMICAL	ALUM
ADDITION POINT	lagoons or filter influent

* Based on design flow/total surface area

TABLE A2.5

EFFLUENT QUALITY PARAMETERS FOR SCHOMBERG WPCP

PARAMETER	BASIS		CONCENTRATIONS (mg/L)	LOADING (kg/yr)
BOD (mg/L)	monthly	Apr	20.00	
		May	10.00	
		Jun	6.00	
		Oct	6.00	
		Nov	10.00	
TSS (mg/L)	monthly		15.00	
TP (mg/L)	monthly		0.30	75.00
Total NH3-N (mg/L)	monthly	Apr	8.00	
		May	2.50	
		Jun	1.00	
		Oct	1.00	
		Nov	3.50	

The operating authority is experimenting with the filters, but they predict that they will only be loaded in the spring until the treatment facility experiences higher flows. Storage will be used for the rest of the year.

A REVIEW OF OPTIONS TO UPGRADE
WASTEWATER TREATMENT LAGOONS IN SMALL MUNICIPALITIES

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March 1992

A REVIEW OF OPTIONS TO UPGRADE

WASTEWATER TREATMENT LAGOONS IN SMALL MUNICIPALITIES

by: Sherwood C. Reed, P.E., E.E.C., Norwich, VT, USA

Background

This review is intended to be part of an assessment of approaches to upgrade lagoons in small municipalities in Ontario. Two specific processes are the major focus of the larger study; the Sutton Process, and the New Hamburg Process. Both of these processes have been in operation in Ontario for up to ten years, and seem to offer promise for application at other lagoon systems. This review is intended to qualitatively evaluate other lagoon upgrading alternatives which may have application to Ontario.

The main issues of concern, requiring correction by an upgrading technique, are:

- High suspended solids (SS) in lagoon effluents due to algae.
- Seasonal fluctuations and variability in performance affect effluent limits for BOD and SS.
- Significant levels of ammonia (NH_3) in lagoon effluents can be toxic to aquatic life in the receiving waters.
- Significant levels of un-ionized H_2S in spring and winter discharges causing odour and toxicity problems.

The technologies discussed in this report include: Intermittent Sand Filtration, Rock Filters, Constructed Wetlands with a free water surface (FWS), Constructed Wetlands using subsurface flow (SF), Aquaculture, and Land Application, using the slow rate process (SR). All of these concepts, except aquaculture have been in general use in the United States for polishing lagoon effluents for at least five years, with a number of facilities in a variety of locations. The assessment of each technology will focus on the potential for cold climate operation and on the capability to correct the four issues of concern listed above.

INTERMITTENT SAND FILTERS

Intermittent sand filtration is a low cost (depending on availability of land) method for polishing lagoon effluents. This process applies lagoon effluent on a periodic or intermittent basis to the surface of a sand filter bed. As the wastewater passes through the sand, suspended solids and organic matter are removed through a combination of physical straining and biological degradation. This process is similar to slow sand filtration used for drinking

water treatment, and to slow sand filtration of raw sewage which was practiced in the early 1900's. This concept is apparently similar to the "New Hamburg Process" which is evaluated in other sections of this report.

In the United States, the filter surface is typically flooded once or twice per day with lagoon effluent. The influent system should be capable of applying the total daily hydraulic load in less than 6 hours to insure maximum head development and maximum bed reaeration after drainage. The particulate matter collects in the top 5 to 8 cm of the sand and this accumulated material eventually clogs the pore spaces in the sand. The filters are considered plugged and ready for maintenance when the liquid from the previous dose has not dropped below the filter surface. At this time the filter is taken out of service and the top layer of sand removed. The removed sand is either discarded or washed and reused as replacement sand in the future. The length of the filter run is controlled by the size of the sand, the hydraulic loading rate, and the suspended solids concentration in the lagoon effluent. Night time applications to the bed, during periods of high algae content in the lagoon have been shown to be advantageous in extending filter runs. Typical expectations for the length of filter run may range from 30 days with lagoon effluent TSS ≥ 50 mg/l to one year with low lagoon effluent solids. To allow flexibility for cleaning, all systems should have at least two filter beds and three are preferred.

The effluent quality from the filter is totally dependent on the sand size selected for use. The smaller sand sizes produce the best effluent but also clog more rapidly and increases maintenance requirements and costs. A medium sand size (effective size 0.2 - 0.3 mm) can produce an effluent with BOD and TSS ≤ 30 mg/L during a filter run of reasonable duration in a single stage filter bed. More stringent effluent requirements (BOD/TSS ≤ 10 mg/L may require filters in series with larger sand (0.6 - 0.7 mm) in the first unit, and smaller sand (0.15 - 0.4 mm) in the second unit. Most of the operational sand filter beds are the single stage type.

The initial depth of the sand in the filter bed is typically 0.9 m. That is to allow removal of the top 2 - 5 cm layer during each cleaning cycle and replacement of that sand about once per year. The filter should not be operated with less than 0.5 m of sand on the bed. The sand layer is typically underlain by a graded gravel layers to prevent intrusion of sand into the underdrain piping. A typical gravel bed might be 0.3 m deep and contain: 10 cm of pea gravel (6 mm) on top, 10 cm of 20 mm gravel in the middle, and 10 cm of 30 mm gravel on the bottom. The underdrain piping is typically perforated PVC pipe 15 cm in diameter, and connected to an outlet manifold. This manifold should be designed to allow complete drainage of the underdrain network so that air can circulate through the drain system into the filter bed. The base of the filter bed is typically lined, with clay, or membrane liners to prevent infiltration to the groundwater. A dosing basin with a siphon or electrically actuated valves and timer controls are typically used to apply the lagoon effluent to the filter bed. Routine maintenance includes removal of vegetation on the filter surface and periodic raking and removal of the top layer of clogged sand.

When site topography permits, gravity flow or automatic dosing siphons are possible for application of lagoon influent. The use of pumps is typically necessary when filter beds are operated in series to lift the effluent to the

second stage filter unit. The containing walls for the filter unit are typically earthen embankments, but concrete or other materials can be used for smaller systems where space is limited. Washing and reuse of the sand is feasible when local sources of low cost sand are not available.

There are at least 30 intermittent sand filter units in operation in the U.S. and a number of these have been in successful use for over 10 years. The technology is well understood since it derives from slow sand filtration of drinking water which has been practiced for over 100 years.

The major application has been polishing of municipal wastewater lagoon effluents with unacceptable effluent TSS levels due to the presence of algae. The major limitations are the land area required for construction of the system, and the need to periodically remove and replace the upper layers of sand on the bed, and to either clean or dispose of the removed sand. There are also limitations on operation of the filter unit during the winter in cold climate locations. Seasonal operation is possible to avoid these problems. In some locations a floating ice layer is utilized as a protective thermal barrier to prevent complete freezing of the upper portion of the bed. However, this ice layer can inhibit reaeration of the media in the bed and nitrification of NH_3 may be limited in the winter due to lack of oxygen and low temperatures. Very effective removals of BOD and SS can be sustained throughout the winter months on these systems if complete freezing is prevented.

The single stage filter with an appropriate sand size can reliably produce and effluent $\leq$ 30 mg/L for BOD and SS. Sand filters in series can be expected to produce an effluent $\leq$ 10 mg/L for both BOD and SS. Although designed primarily for BOD and TSS removal the sand filter bed may also provide significant ammonia removal via nitrification, if the bed is maintained in an aerobic condition. It is believed that phosphorus removal by these systems is primarily due to entrapment of particulate matter and the related phosphorus, although there may be some adsorptive capacity depending on the iron and aluminum oxides associated with the sands used in the system.

The U.S. EPA studied three of these systems in the late 1970's at Mt. Shasta, CA, Moriarty, NM, and Ailey, GA. Three sampling events of 30 d duration each, occurred on a seasonal basis at each site. The basic design criteria for two of these three systems are given in Table 1.

Table 1. Design Criteria for Intermittent Sand Filters in EPA Study.

Parameter	Mt. Shasta, CA	Ailey, GA
Lagoon type	Aerated	Facultative
Flow (m^3/d)	4542	303
Total filter area (ha)	2.4	0.11
Number of filters	6	2
Sand depth (m)	0.61	0.76
Effective size (mm)	0.37	0.50

Tables 2 and 3 summarize the seasonal performance data from the sites in California and Georgia during the EPA study.

Table 2. Performance Data, Mt. Shasta, CA

Parameter	Effluent					
	Winter		Spring		Summer	
	Lagoon	Filter	Lagoon	Filter	Lagoon	Filter
Temperature (°C)	5.8	5.7	11.0	10.7	24.4	23.8
pH	8.3	6.4	8.5	7.1	9.3	7.0
BOD (mg/L)	26	21	20	7	19	4
SS (mg/L)	37	26 30%	33	11 66%	69	13 80%
TKN (mg/L)	10.8	5.7	11.5	8.5	16.3	8.6
NH_3 (mg/L)	7.6	4.0 50%	4.4	0.4 90%	4.1	0.4 9[illegible]
T P (mg/L)	3.4	3.0	2.5	1.9	5.1	3.8
Fecal Coli (#/100)	720	53	179	37	20	2.4

Table 3. Performance Data, Ailey, GA.

Parameter	Effluent					
	Winter		Spring		Fall	
	Lagoon	Filter	Lagoon	Filter	Lagoon	Filter
Temperature (°C)	9.5	10.8	19.9	19.2	26.1	25.6
pH	8.6	7.1	8.7	6.8	9.5	7.3
BOD (mg/L)	34	14	11	4	20	5
SS (mg/L)	49	19 60%	31	14 55%	48	11 ~~88%~~ 75%
TKN (mg/L)	10.4	6.3	2.8	1.2	8.8	4.8
NH_3 (mg/L)	1.5	1.1 30%	0.4	0.05 0.1	0.05	0.08
T P (mg/L)	4.0	3.6	1.4	1.3	3.8	3.1
Fecal Coli (#/100)	149	21	8	3	9	1

Some seasonal effects are apparent in both Tables 2 and 3, particularly for BOD and NH_3. However, neither site is close to the probable expected winter water temperatures in Ontario, so the winter removals shown in Table 2 for the California site will probably be less effective in Ontario.

The following criteria are applicable for design of a single stage filtration system. At least two filter beds, each designed to receive the total flow should be used.

Hydraulic loading rate: 480 L/d/m^2, using two or more equal dosings per day.

Depth of media: Large gravel 10 cm, medium gravel 10 cm, Small gravel 10 cm, sand 0.9 m.

Sand size: 0.15 mm to 0.30 mm effective size, uniformity coefficient <7. Concrete sand meeting the AASHO M6 specifications for highway construction

would be acceptable.

Influent zone: should have a gravel splash pad using 5 cm gravel.

Underdrains: Maximum spacing 1.5 m, minimum lateral size 15 cm diameter.

Cleaning frequency: 1 month to >1 year depending on hydraulic loading and TSS concentrations.

Preliminary costs for planning purposes can be estimated with the following procedures:

1. Design basis: Single stage sand filter used to produce an effluent BOD/TSS of 30 mg/L. Two filter beds, in parallel to be constructed at the site. Each filter unit sized for full design flow, loading and materials criteria as listed above. Sand removed from bed taken to landfill. Costs based on November 1990 prices ($US) (ENR = 4780).

2. Construction costs include excavation when necessary, membrane liner, gravel and sand layers as specified above, gravity delivery, and underdrain network. Costs do not include land, or any pumping.

3. The equations given below are valid for a preliminary cost estimate for design flows up to 19,000 m^3/d.

C = Costs, in millions of dollars ($US)

Q = Wastewater flow in Mgal/d (US)

CONSTRUCTION COSTS	OPERATION AND MAINTENANCE COSTS
$C = 0.600(Q)^{1.031}$	$C = 0.065(Q)^{0.743}$

Potential Application in Ontario

In summary, based on operational experience in the U.S. it should be possible to operate an intermittent sand filter system in Ontario on a year-round basis. Effective removal of BOD and SS should be possible regardless of season. Ammonia removals can be very effective during the warm months of the year but not during the coldest part of the winter. Phosphorus removal appears to be negligible regardless of season. If a lagoon discharges significant concentrations of un-ionized H_2S during the winter months there are likely to be objectionable odours at the filter unit. If the filter unit is maintained in an aerobic state on a year-round basis there should not be an adverse impact on receiving waters from un-ionized H_2S. If year-round control of both NH_3 and H_2S are required it may be necessary to operate the filter system on a seasonal basis.

FREE WATER SURFACE CONSTRUCTED WETLANDS

Free water surface (FWS) wetlands used for the treatment of wastewater are designed and managed so the water surface is exposed to the atmosphere. FWS systems in operation include both natural and constructed wetlands. The operational natural systems include swamps (with trees the dominant vegetation), marshes (with grasses and other emergent vegetation dominant), and bogs. All of the FWS constructed wetlands are the marsh type. The submerged portions of the vegetation, the benthic detritus, and the soil surface serve as the substrate for attached bacteria, protozoa, and other organisms which are believed to be primarily responsible for the wastewater renovation occurring in these systems. The major source of oxygen to support this biological activity is reaeration occurring at the water surface. These systems can be designed for secondary treatment, advanced secondary treatment, or tertiary polishing. The land area increases significantly as the treatment level requirements become more stringent. The use of natural wetlands for wastewater treatment is limited to relatively few examples in the U.S. In these cases, any discharge to the wetland must satisfy NPDES limits, so these wetlands are typically used for advanced wastewater treatment (AWT) or tertiary polishing.

A survey in 1990 documented over 80 FWS constructed wetlands in numerous States ranging from North Dakota to central Florida. The largest system treats 76,000 m^3/d, the smallest 57 m^3/d. About 70 percent of these systems treat 4000 m^3/d or less. Operational modes vary from continuous flow to seasonal operations where effluent is stored in the lagoon during the winter and discharged to the wetland in the warm months.

The most commonly used vegetation in the constructed FWS wetlands includes cattail (*Typha*), bulrush (*Scirpus*), and reeds (*Phragmites*). About 30 percent of the operational FWS systems planted only cattails. Harvesting of these plants is not routinely practiced in the U.S. since plant uptake of nitrogen or phosphorus is not considered to be a significant factor. The major pathway for nitrogen removal in these systems is believed to be biological nitrification followed by biological denitrification. Phosphorus removal depends on interactions with the soil and precipitation in the benthic material. Harvesting, and removal of dead plant material may be needed on an infrequent schedule in these constructed wetlands if the accumulation of material interferes with flow distribution in the system. The water depth in these systems range from 5 cm to about 1 m; typical water depths in the constructed type range from 0.3 to 0.6 m.

A major advantage for the constructed wetland is the improved hydraulic control since the system is constructed to a specified grade to insure uniform flow throughout. In the natural wetland, any existing flow channels will continue in active use so uniform flow cannot be insured. In both constructed and natural systems, the wastewater to be treated is typically introduced via a multiple port manifold at the head of the system. Effluent collection is accomplished with either a manifold or an effluent weir structure. It is recommended that this discharge structure, particularly for constructed systems, include some method for controlling the water level in the system, and for draining the wetland bed when necessary for maintenance.

Some form of preapplication treatment is necessary in all cases. The minimal acceptable level for constructed wetlands is the equivalent of primary, with either septic or Imhoff tanks for smaller systems. About 41 percent of the constructed FWS systems utilize facultative lagoons for preliminary treatment. Some of the largest systems are designed for tertiary polishing and receive effluent from a mechanical AWT plant. Other variations, in the more arid portions of the country include total retention (no discharge) systems where the liquid is lost through a combination of seepage and evapotranspiration. Seasonally operated FWS wetlands are used in very cold climates. In these cases, the wastewater is retained in a lagoon during the winter months and discharged at a controlled rate to the wetland during the warm summer months.

The use of FWS wetlands, particularly the constructed type, has increased significantly since 1980, with most of the increase occurring since 1988. The systems are widely distributed in the U.S. and can be found in about 20 states. A general consensus has not developed regarding design of these systems. Preliminary, first order design models are available for BOD removal. Regression equations for removal of ammonia and total nitrogen are also available, but their reliability is uncertain since they do not reflect the oxygen status and seasonal temperature conditions in the wetland unit. Additional work is needed to optimize the model for BOD removal and to more accurately define the criteria for nitrogen and phosphorus removal.

These wetland systems can be utilized for effective removal of BOD, COD, TSS, and with sufficient detention time for nitrogen and phosphorus removal. Heavy metals seem to be removed effectively and about a one log reduction in fecal coliforms can be expected. The majority of the systems in use follow facultative lagoons. The concept has been successfully used in combination with the overland flow (OF) land treatment concept. FWS wetlands are also in current use treating storm water, acid mine drainage, pulp and paper wastewater and other industrial operations.

A major limitation, affecting the removal of nitrogen in these wetlands, is the availability of oxygen in the system. This oxygen is essential to support the biological nitrification of ammonia. There is, typically sufficient available carbon in anoxic zones to support denitrification and thereby almost completely remove nitrate nitrogen from the wastewater stream. Apparently, there is insufficient atmospheric reaeration at the water surface to supply the necessary oxygen when the water is more than a few inches deep. The emergent aquatic plants used in these systems have the capability to transmit oxygen from their leaves to the roots but any excess oxygen from this source is lost in the soil substrate and is not available to the wastewater. Several systems have been constructed with intermediate deep water trenches (± 1 m) to provide open water for reaeration and to support the fish who prey on mosquito larvae. A number of these open water trenches have become covered with a dense layer of duckweed and their effectiveness for oxygen transfer has not been well documented.

An apparently successful approach combines the FWS wetland with the OF land treatment concept, in a multiple cell combined system. The water depth, in each cell in the series varies from ± 1 cm at the head of the cell (the OF zone) to 0.3 - 0.6 m at the end of the cell (the FWS zone). The shallow flow in the OF zone permits active reaeration and nitrification, with denitrification

apparently occurring in the anoxic portions of the FWS zone. Another promising approach would utilize a bed (about 0.6 m deep) of gravel placed near the head of the wetland cell. Effluent would be recycled to this bed with sprinklers and as the liquid percolates vertically nitrification should occur. The nitrified effluent would then mix with the partially treated wastewater in the FWS cell and denitrification could occur in the anoxic zones. This approach has been recommended for several systems in Kentucky and Louisiana.

Temperature can also be a limitation in colder climates by reducing reaction rates for BOD removal and inhibiting biological activity for nitrification and denitrification. Winter ice formation on the water surface will also limit atmospheric reaeration. As a result of these constraints, many systems in South Dakota, and several proposed for Whitehorse, Teslin, Haines and other communities in the Yukon would operate on a seasonal basis with discharge to the wetland component during the summer months. Mosquito development can also be a problem, but control is possible with fish that prey on mosquito larvae and with appropriate insecticides. Mosquito development will be no worse than already occurring on natural marshes in the area.

A significant limitation on the use of natural wetlands is related to the area of the system which will actually be effective for wastewater treatment due to the presence of channeling and "dead spots." A careful survey to establish water depths and profiles, and a tracer study are necessary to define hydraulic characteristics and resident time in the natural systems. A conservative, preliminary "rule of thumb" estimate would suggest that 10 percent or less, of the total area will be effective for treatment.

The aspect ratio (L:W ratio) of the FWS wetland cell can strongly influence performance due to the potential for short circuiting of flow through the system. Studies in Listowel, Ontario and elsewhere suggest the use of an L:W ratio of at least 5:1 for optimum performance. A cell with a low aspect ratio will not perform as well as a long and narrow unit unless there are other methods for flow redistribution within the system. These redistribution methods might include intermediate open water trenches, or internal dikes with a manifold for flow redistribution.

Natural wetlands may be utilized in their existing state with the construction of appropriate inlet and outlet structures. Constructed wetlands require careful excavation and grading to insure uniform flow; a liner or other impermeable barrier may be necessary, and establishment of appropriate vegetation and suitable inlet and outlet structures are required. Pumps may be needed to deliver the wastewater to the site, and for internal recycle if necessary for ammonia removal. Disinfection and post aeration facilities may also be needed, depending on local requirements.

FWS wetland systems can produce an effluent BOD and TSS of less than 10 mg/L, total N less than 3 mg/L, and total P less than 1 mg/L depending on the size of the system and the composition of the influent wastewater. At Lakeland, FL the 566 ha combined overland flow constructed wetland system receives 53,000 m^3/d from advanced secondary treatment plant and produces a final effluent with BOD 2 mg/L, TSS 5 mg/L, NH_3-N 0.3 mg/L and total N 2.0 mg/L (Influent to the wetland: BOD 10 mg/L, TSS 13 mg/L, NH_3-N 6.5 mg/L, TN 15 mg/L). The 486 ha wetland system in Orlando, FL, polishes the 76,000 m^3/d effluent from the Iron Bridge AWT plant and produces a final effluent with BOD 2 mg/L, TSS 4 mg/L, TN

0.9 mg/L, TP 0.08 (Influent to the wetland: BOD 5 mg/L, TSS 11 mg/L, TN 5 mg/L, TP 0.7 mg/L). Most constructed FWS systems are only required to achieve secondary or advanced secondary levels (some with NH_3) removal) and can satisfy these requirements using higher hydraulic loadings and smaller land areas than the systems described above. It is not possible to achieve zero BOD or TSS in these wetlands due to the presence of naturally occurring organics in the systems. In most cases, a final BOD of ≥ 5 mg/L is the best performance which can be achieved.

At least 40 FWS constructed wetland systems are used in the State of South Dakota for polishing and upgrading facultative lagoons, and most of these are seasonal operations with discharge to the wetland in the warm season only. In some cases the wetland component is designed for no final discharge in normal years with seepage and evapotranspiration accounting for the water applied. In "wet" years the discharge period to a receiving water might range up to 90 days depending on the number of rainfall events. Table 4 summarizes performance data from two of these systems.

Table 4. Performance Data from Seasonally Operated Wetlands in South Dakota

Parameter	Huron, SD*		Arlington, SD$^{\Phi}$	
	Influent	Effluent	Influent	Effluent
BOD (mg/L)	6 to 19	4 to 22	14	11
SS (mg/L)	3 to 80	4 to 40	85	81
NH_3 (mg/L)	13 to 19	0.7 to 2.7	0.13	0.08
TKN (mg/L)	-	-	3.3	2.7
Total P (mg/L)	-	-	0.85	0.63
Fecal Coli (#/100)	-	5 to 160	520 to <10	40 to <10

* Discharge period, June 1991.

Φ Discharge period, October 1989.

Constructed wetland systems designed for tertiary polishing of AWT or advanced secondary effluents have between 6 and 8 ha of effective treatment surface per 1000 m^3/d of design flow. Constructed systems designed for advanced secondary treatment and low levels of NH_3 (≤ 5 mg/L) typically have 2 to 4 ha of effective surface area per 1000 m^3 of design flow. Systems designed for just secondary treatment (no NH_3 removal) typically have between 0.5 and 1.5 ha per 1000 m^3 of design flow, depending on the quality of the influent to the system. The use of these estimates for natural wetlands must take into account the actual effective treatment area which is available, because of natural channeling this effective area may be ten percent or less of the total area available.

These systems can be very reliable for removal of BOD, TSS and nitrogen if designed and operated properly. The capability for phosphorus removal seems to have a finite limit. A conservative estimates of the long term phosphorus removal capability of these FWS wetlands is about 15 kg/ha/yr. Because of the relatively short operational life of existing constructed systems, little is known regarding the long term requirements for cleaning and renovation of the wetland bed. However, the only reason this may be required is to maintain uniform flow conditions in the system.

Cost estimates for very preliminary planning purposes can be developed using the following procedure.

1. Design basis: used for advanced secondary treatment (BOD & SS only), average detention time = 5 days, average depth 1 ft. Costs based on November 1990 prices (ENR = 4780).

2. Construction costs include excavation, grading and other earthwork, service roads, stocking vegetation. Costs do not include land or pumping. Operation and maintenance costs do not include costs for harvesting or disposal/utilization of harvested materials.

3. These equations are valid for a very preliminary cost estimate for design flows up to 76,000 m^3/d.

4. The cost of land is a very significant factor in total project costs, but is very site specific so an estimate based on local sources is essential.

C = Costs, in millions of dollars ($US)

Q = Wastewater flow in Mgal/d (US)

CONSTRUCTION COSTS	OPERATION AND MAINTENANCE COSTS
$C = 0.233(Q)^{0.765}$	$C = 0.009(Q)^{0.671}$

Potential Application in Ontario

It should be possible to operate a free water surface wetland in the southern portion of Ontario on a year round basis for lagoon upgrading for both BOD and SS removal. Work conducted by the Ontario Ministry of the Environment at Listowel, Ontario, has already established that capability. Such wetlands are designed to form an ice cover during the winter months with maintenance of flow beneath that ice. The low levels of oxygen and the low water temperatures beneath the ice will restrict NH_3 removal in the winter months. Removal of phosphorus is negligible, requiring very large land areas for effective long term removal. Winter or spring discharge of un-ionized H_2S from a lagoon to an ice covered FWS wetland would not improve this condition and the wetland itself may contribute to the H_2S and NH_3 load on the receiving waters. If stringent NH_3 and H_2S limits prevail on a year-round basis then a seasonal operation would be recommended for Ontario climate conditions.

SUBSURFACE FLOW CONSTRUCTED WETLAND

Subsurface Flow (SF) constructed wetlands used for the treatment of wastewater are constructed as a bed or channel containing up to 0.6 m of appropriate media (rock, gravel, sand, and other soils have all been used). The media is planted with emergent vegetation commonly used in marshes and the water level in the bed is designed to remain below the surface of the media. This configuration is thought to have several advantages over natural marshes and free water surface (FWS) constructed wetlands. The presence of the media provides a greater surface area for development of attached growth microbial activity, which is believed to be the major factor responsible for treatment in these systems. In addition, because the water level is supposed to remain below the surface of the media there should be no problems with mosquitoes, odors, or public access. Because of the potentially higher microbial reaction rates, the SF wetland can be smaller and less costly than the FWS type for the same flow rate and effluent goals.

The SF concept is also called a vegetated submerged bed (VSB), a rock reed filter, the root zone method, and hydrobotanical system. The lack of consensus on a title is also reflected in the lack of consensus in their design, construction, and operation. Most of the systems are designed for secondary or advanced secondary treatment, with an increasing number having NH_3 removal requirements. A survey in 1990 documented over 80 SF constructed wetlands in planning, construction or operation in the U.S. The largest system in operation treats 11,000 m^3/d, the smallest 4 m^3/d. The median capacity of the operational systems is 230 m^3/d, with 90 percent treating less than 3800 m^3/d.

The most commonly used vegetation in these systems are Cattails (*Typha*), bulrush (*Scirpus*), and reeds (*Phragmites*). About 40 percent of the SF systems use bulrush, with the remainder dependent on cattails, reeds, or combinations of other plants. The plants are not considered to be a major factor in nutrient removal via plant uptake. Their major contribution is to serve as a substrate for microbial development and as an oxygen source in the root zone. These emergent aquatic plants have the capability to transmit oxygen to their root systems. This oxygen does not diffuse widely through the soil or other media, but is believed to be available at microsites on the root structure of the plants. This is the major oxygen source for optimum BOD removal and for nitrification within the media profile. Biological denitrification can then occur in anoxic portions of the bed. Phosphorus removal is limited to interactions with the supporting soil, or with the media if soil is used. Soil beds are not used in the U.S. because of their very limited hydraulic capacity. Most soil beds in Europe experience significant surface flow for this same reason. The media used in the U.S. ranges from fine gravel (≥ 0.6 cm) to large crushed rock (≥ 15 cm), 1.3 to 5 cm sizes are most typical. The depth of the media ranges from 0.3 to 1 m with 0.6 m being most common.

Some form of preapplication treatment is necessary in all cases. The minimal acceptable level for constructed wetlands is the equivalent of primary, with either septic or Imhoff tanks for smaller systems. About 44 percent of the constructed SF systems in the U.S. utilize facultative lagoons for preliminary treatment. Most of these added the SF wetland bed to improve performance of the existing facultative lagoon. A number of the smaller systems utilize septic or Imhoff tanks for preliminary treatment. A number of systems follow the SF

component with disinfection and post aeration in response to local requirements.

Many of the early systems were constructed with a high aspect ratio (L:W) of 10:1 or more, based on the assumption that it was necessary to maintain plug flow conditions. There was insufficient hydraulic gradient to maintain subsurface flow in these very long cells and surface flow can be commonly observed on many of them. Many of the newer systems have been designed with an L:W of 2:1 or less and also incorporate adjustable outlet works to permit control of the water level in the bed.

The use of SF constructed wetlands has increased significantly since 1980, with most of the increase occurring since 1988. Most of these are concentrated in the Gulf states, Arkansas, Kentucky and Tennessee, since that is where the initial developmental work occurred. A general consensus has not developed regarding design of these systems. Preliminary, first order design models are available for BOD removal. Regression equations for removal of ammonia and total nitrogen are also available, but their reliability is uncertain since they do not reflect the oxygen status and seasonal temperature conditions in the wetland unit. Additional work is needed to optimize the model for BOD removal and to more accurately define the criteria for nitrogen and phosphorus removal.

These wetland systems can be utilized for effective removal of BOD, COD, TSS, and with proper design and operation for nitrogen. Phosphorus removal will be limited during the relatively short detention times available in the SF system. Heavy metals seem to be removed effectively and about a one log reduction in fecal coliforms can be expected.

A major limitation, affecting the removal of nitrogen in these SF wetlands, is the availability of oxygen in the media profile. This oxygen is essential to support optimum BOD removal and the biological nitrification of ammonia. There is, typically sufficient available carbon in anoxic zones to support denitrification and thereby almost completely remove nitrate nitrogen from the wastewater stream. As indicated previously, the plant roots are thought to be the major source of oxygen in the profile and the original expectations were that these roots would spread throughout the entire bed profile. Recent investigations have shown that the plant roots in operational beds, up to four years old, only penetrate to a depth of less than 0.3 m. Since the bed is usually at least 0.6 m deep, the lower portion must be anaerobic with no oxygen to support nitrification in that zone.

It will be essential for the future utilization of this technology to develop methods for improving the oxygen supply and the conditions necessary for nitrification. Several methods have been proposed including varying the water level in the bed to induce root penetration, alternating flow in parallel channels, on a frequent basis, to allow atmospheric reaeration of the media, and zones of open water with mechanical aeration if necessary. The most promising would impose a bed of fine gravel on top of the SF media, near the head of the system. Effluent would be recycled to the gravel bed with sprinklers, and nitrification should occur as the liquid percolates vertically through the fine gravel. The nitrified liquid will then mix with the partially treated wastewater in the SF media and denitrification can occur in the

available anoxic zones. This approach has been proposed as a retrofit for several systems in Kentucky and Louisiana.

The hydraulic design of many of the early systems was inadequate. Since the intended flow path is through a porous media it is necessary to provide sufficient hydraulic gradient to sustain the desired subsurface flow. The maximum available hydraulic gradient is dependent on the depth of the media and the length of the bed. In a number of systems with a high L:W ratio there is insufficient gradient available to maintain subsurface flow at the design rate. In a number of systems the effluent manifold was placed in the upper part of the bed so it was not possible to develop the potential gradient. Future designs should use shorter bed lengths (L:W $\leq$2:1) and provide a method for controlling the water level in the bed.

Temperature can also be a limitation in colder climates by reducing reaction rates for BOD removal and inhibiting biological activity for nitrification and denitrification. Winter freezing of the bed is unlikely in most of the contiguous U.S. as long as the system is operated continuously, and there is a layer of vegetation on the bed surface to provide insulation.

Many of the early systems exhibit surface flow on a continuous basis. This was originally thought to be due to clogging by the accumulation of organic solids in the system. It is now believed that most of these cases of surface flow are due to an inadequate hydraulic gradient in the system. There are some cases of local clogging of up to 25 percent of the bed length. Investigations have shown that this clogging material is largely inorganic in character and may be related to construction activities. Theoretical calculations based on the accumulation of wastewater detritus indicate a useful life of 50 to 100 years before clogging is a concern.

The hydraulic design and performance expectations assume that the bed will be carefully graded and the media carefully placed so that uniform flow will prevail throughout the media during operation. Tracer studies of a few operational systems indicate severe short circuiting of flow is occurring. Inadequate control during bed construction and media placement is believed responsible. The soil bottom should be graded and compacted to the same degree required for a highway subgrade to withstand the stresses induced by the trucks delivering the media, and access should be restricted during wet conditions. Clean, washed media should be used if possible, and the lightest possible equipment used to spread the material in the bed.

SF wetland systems can produce an effluent BOD and TSS of less than 10 mg/L, depending on the size of the system and the composition of the influent wastewater. At Mandeville, LA, the 0.16 ha SF wetland receives about 57,000 m^3/d from a partial mix aerated lagoon, and produces an final effluent with BOD 7 mg/L, TSS 7 mg/L, NH_3 2 mg/L, NO_3 0 mg/L Total N 3.0 mg/L, Total P 2 mg/L (Influent to the wetland: BOD 35 mg/L, TSS 72 mg/L, NH_3 1 mg/L, NO_3 3 mg/L, Total N 6 mg/L, Total P 2 mg/L). The detention time in the SF bed is about 1.3 d. The complete removal of NO_3 in this system is evidence of effective denitrification under the anoxic conditions prevailing in the bed. The increase in NH_3 at this system and numerous others is believed due to the decomposition of trapped algae and other organic detritus under anaerobic conditions. As

indicated previously, modifications to the SF concept will be necessary to provide the necessary oxygen to support the desired oxidation reactions.

As described previously, preliminary design models are available for BOD, NH_3, and TN removal. These models require further verification and optimization. System design should commence with selection of the media and related hydraulic considerations. To insure an acceptable safety factor, a fraction of the potential hydraulic conductivity (permeability) of the media should be used for design. Systems designed for secondary or advanced secondary treatment typically have between 0.5 and 1 ha of SF bed area per 1000 m^3/d of design flow, depending on the characteristics of the wastewater.

Most of the existing systems expected effective NH_3 removal but were not specifically designed for that purpose. In the majority of cases, these expectations have not been realized. One successful system has a 0.3 m depth of pea gravel as the media and provides about 2.7 ha per 1000 m^3 of design flow (influent from activated sludge package plant). Use of the previously described recirculating gravel filter in combination with the SF wetland bed also seems to be a promising method for effective NH_3 removal.

A cost estimate for very preliminary planning purposes can be developed using the following procedures.

1. Design basis: used for advanced secondary treatment (BOD & SS only), average detention time ± 3 days, media depth 0.6 m. Costs based on November 1990 prices (ENR = 4780).

2. Construction costs include excavation, grading and other earthwork, purchase and placement of the media, service roads, planting vegetation. Costs do not include land or pumping, Operation and maintenance costs do not include costs for harvesting or disposal/utilization of harvested materials, or management of any preliminary treatment facilities.

3. These equations are valid for a very preliminary cost estimate for design flows up to 20,000 m^3/d.

4. The cost of land is a very significant factor in total project costs, but is very site specific so an estimate based on local sources is essential.

C = Costs, in millions of dollars

Q = Wastewater flow in Mgal/d (US)

CONSTRUCTION COSTS	OPERATION AND MAINTENANCE COSTS
$C = 0.184(Q)^{0.765}$	$C = 0.009(Q)^{0.671}$

Potential Application in Ontario

It should be possible to operate an SF constructed wetland in much of Ontario on a year round basis for successful BOD and SS removal. As presently designed and configured (ie: 1 to 3 day HRT, single 0.6 m deep bed) they will not provide acceptable NH_3 removal in Ontario or anywhere else, during the winter or summer months.

The concept has the potential for effective NH_3 removal but modifications will be necessary. These might include either use of shallow beds or inducing deeper root penetration to provide oxygen from the plants to the entire profile. The retention time in this type of system might have to be extended to 8 to 10 d and there might be seasonal limitations on the oxygen transfer capability by the vegetation.

Another alternative is to construct the system in multiple cells or stages and utilize the final cells for nitrification/denitrification. This would require a supplemental oxygen source developed through the use of cascades, blowers, pumps, sprinklers or other mechanical equipment. The HRT in such a system might be 1 to 3 days as compared to the 8 to 10 days if the vegetation is the sole source of oxygen.

The discharge of un-ionized H_2S from an ice covered lagoon to the type of SF wetland described above might not achieve significant benefits due to the general lack of sufficient oxygen in the deeper parts of bed. The anaerobic portions of the bed may actually contribute additional NH_3 and H_2S. Odours should not be a problem in the vicinity of the wetland unit since the water level is maintained below the media surface and the top portion of the bed should be aerobic. Odours and toxicity problems may be a factor at the point the SF wetland discharges to the receiving waters.

A seasonal operation of the SF wetland concept should avoid winter problems with NH_3 and H_2S, if the modifications for enhanced oxygen status are incorporated in the design. This type of wetland has advantages for locations in relatively close proximity to habitations because of the odour and mosquito control inherent in the concept.

ROCK FILTERS

The rock filter has been used as a method for removal of algae from lagoon effluents. The system consists of a submerged bed of rocks (rock size typically 8 to 20 cm), through which lagoon effluent is passed horizontally or vertically. The algal solids are expected to settle on the rock surfaces and in the void spaces and then undergo biological decomposition. The system can usually produce a final effluent with BOD_5 and TSS less than 30 mg/L. The system is not as successful, or as reliable, for the removal of ammonia (NH_3-N). The low cost and simple operation make the concept attractive for small communities if ammonia limits do not prevail.

The concept was developed in Kansas in the early 1970's. There are about 20 operating systems in the U.S. and most of these were constructed between 1970 and 1985. The design flow at these operational systems ranges from 150 to

19,000 m^3/d. New applications of the rock filter concept has diminished in recent y due to the problems with ammonia removal and to the emergence of constructed wetlands concepts for upgrading lagoon performance. However, these new wetland concepts are also having problems with ammonia removal.

Most of the operational systems are designed for horizontal flow with the rock bed placed at or near the effluent end of the final cell in the lagoon system. In some cases the effluent is collected by a manifold buried in the rock bed, and in other cases effluent is discharged from a open water area on the downstream side of the filter bed. These filter beds typically extend about 0.6 m above the maximum water level in the lagoon cell to prevent algae growth on the bed surface. The hydraulic loading rate is typically expressed as a volume of water applied per day, per cubic meter of rock media in the bed. Hydraulic loadings have ranged from 270 to 1200 L/d/m^3.

Potential Application in Ontario

The rock filter is not considered to be an acceptable lagoon upgrading technology for application in Ontario since it can only produce marginally successful results for BOD and SS removal. It also does not have the capability to remove NH_3 or H_2S when operated on either a seasonal or continuous basis.

A second stage, deep rock (or plastic media) filter bed with either intermittent or unsaturated flow in the vertical direction, and possibly recycle, should support suitable microorganisms and have sufficient oxygen to deal effectively with both NH_3 and H_2S. This, in effect, reinvents the trickling filter for use as a nitrification/oxidation bed. Such a concept could work in Ontario if the bed were covered to prevent freezing and if the BOD and SS concentrations in the lagoon effluent were reduced in a preliminary step.

AQUACULTURE CONCEPTS

A variety of aquaculture concepts for waste treatment have attracted a great deal of interest in recent years. In general these combine microorganisms, protozoa, and higher plants and animals in aerated tanks or channels. In cool climates the system is typically enclosed in a glasshouse for thermal protection and to maximize the utilization of solar radiation. One such concept is called "Solar Aquatics." Performance claims for these systems range from at least tertiary treatment to an effluent equivalent to drinking water in quality, as well as beneficial use of the plants and animals harvested from the system, but very limited data have been published to support these claims.

A system of this type was operated, at pilot scale, at the Sugarbush Ski Resort in Warren, VT during the late 1980's. The author of this report was retained to conduct an independent assessment of performance of this system and the results will be published in July 1992. The system included aerated tanks and channels and a marsh component all enclosed in a translucent plastic greenhouse. The system was operated on a continuous basis from March of 1987 through March of 1989. During part of that period it was used to polish lagoon effluent, during the latter part of the test period untreated wastewater was used. Over 70 species of vegetation were used in the system components, as well as fish, snails and other organisms. The average performance data during the 14

month period (Jan 1988 - March 1989) when untreated wastewater was applied to the system are presented in Table 5.

During some periods the nitrification of NH_3 to NO_3 was excellent and at other times it was marginal. Phosphorus removal was not successful during the entire study period. Methods for improving the level and reliability of performance for these materials are supposedly under development. The average BOD loading was about 184 kg/ha/d; this is a higher surface loading than typically used on most lagoon systems. On a volumetric basis the BOD loading was about 0.02 kg/m^3 of reactor volume and this is significantly less than the typical loading on extended aeration and/or similar activated sludge processes. The average hydraulic residence time in the complete system was slightly over 12 days, at an average flow of 5 m^3/d.

Table 5. Performance Data, Solar Aquatics System, Warren, VT

Parameter	Influent mg/L	Effluent mg/L	% Removal
BOD	220	10	95
TSS	162	6	96
TKN	51	12	77
Organic N	29	4	86
NH_3	22	8	64
NO_3	0.6	9	-
Total N	52	21	60
Dissolved P	5	5	0
Fecal Coli (#/100 ml)	6×10^6	1500	99+

Potential Application in Ontario

Based on the results in Table 5 this aquaculture system could satisfy the Ontario goals for BOD, SS, NH_3, and H_2S, if the reliability has been improved by the system proponents. The total cost for construction and operation of this pilot facility was estimated (by the owner) to be about \$11/m^3 treated. The capital costs for this temporary facility are not applicable for a large scale permanent system.

The capital costs for this type of system are likely to be 5 to 10 times higher than the technologies discussed previously in this report. If located at a site in Ontario the glasshouse structure would have to withstand higher snow loads and extra energy for light and heat would be necessary during the winter months. Operation of the facility would also require more time and a higher skill level than needed for the technologies discussed previously. As a result, even though the concept may satisfy water quality goals it will probably not be economically competitive with other lagoon upgrading alternatives in Ontario.

SLOW RATE LAND TREATMENT

Slow rate (SR) land treatment involves the controlled application of partially treated wastewater to a vegetated land surface. Operational sites

include perennial grasses for hay or pasture, agricultural row crops (corn, soybeans, etc.), forests, greenbelts, parks, and golf courses. The minimal degree of preliminary treatment is primary with the degree of treatment then increasing as the risk of direct public exposure increases. The SR process offers the potential for tertiary levels of treatment with very effective removal of BOD, COD, TSS, nitrogen, phosphorus, metals, pathogenic bacteria, and virus. There are two basic types of SR systems treating municipal wastewaters. Type 1 optimizes the treatment potential of the site by applying the maximum amount of wastewater possible to the land, consistent with design requirements. This requires more permeable soils and either more water tolerant crops or some sacrifice in some crop yields. The type 2 concept optimizes the irrigation potential of the available wastewater resource by applying the least amount of water possible to the maximum possible land area for the production of high market value crops. The type 2 systems tend to be similar to conventional irrigation practice and are usually found in the more arid parts of the country. Either type of system can be developed as a municipally owned and operated system, a municipally owned but contract operated by a farmer, or farmer owned land and contract with municipality for delivery of partially treated wastewater.

Wastewater treatment occurs through a combination of filtration, adsorption, ion exchange, precipitation and microbial action, on and in the soil matrix, and via uptake of nutrients by the vegetation. Most of the treatment responses occur at or near the soil surface and in the root zone for the vegetation, but the physical and chemical reactions continue as the water percolates through the soil profile. The percolate in most systems then becomes part of the local groundwater. As a result, groundwater protection is a major design consideration for SR systems. The typical design approach assures that nitrate (or other critical parameters) in the percolate do not exceed drinking water specifications prior to mixing with the groundwater. This is to insure that the groundwater will meet drinking water limits at the project boundary. Most states have specific requirements or guidelines and these should be considered prior to system design. In some locations, where the groundwater has no potential as a drinking water aquifer the system design may be modified accordingly. In other cases, where the groundwater table is relatively shallow it is possible to install an underdrain network to recover all of the percolate. In this case surface discharge, rather than groundwater, requirements would govern the design.

A variety of methods are available for application of wastewater at the site. These include a number of portable or fixed sprinkler systems, and low pressure or gravity distribution from gated pipe or open ditches. In the latter case, mostly used for agricultural operations in the western states, the excess surface flow is collected in a ditch at the end of the field and this "tailwater" is returned to the head of the system for redistribution. This type of system can either be "ridge and furrow," where the row crops are planted on the ridges and the wastewater flows in the parallel furrows, or the "border strip" type where the water flows over the surface of the entire area. The "border strip" type is used for alfalfa and similar crops. Sprinkler systems are designed for uniform coverage on the field at a rate not to exceed the hydraulic capacity of the soil so runoff is prevented. The most commonly used sprinkler systems are the "solid set" type which typically consists of a network of distribution piping with vertical risers and sprinkler nozzles, and the "center pivot" system, where the sprinklers are fixed to a long elevated

set type; parks, golf courses, etc, typically use the solid set type with "pop-up" sprinklers.

Wastewater applications occur on a repetitive cyclic pattern. This could apply up to 10 cm/wk either in a single application during one day in the week or several smaller doses, depending on the soil characteristics and crop requirements. The wastewater application is typically followed by a resting period for the restoration of aerobic conditions on and in the near surface soil profile. Wastewater applications are curtailed during extreme precipitation events (to avoid runoff), during periods required for crop harvest or site maintenance, during subfreezing weather (± - 4°C), and during the non growing season if agricultural crops are used in the system. The cumulative effect of these non-application periods results in temporary wastewater storage, in ponds, for most SR systems. This storage period might range from a week or so in southern climates with perennial grasses, to several months in colder climates using row crops. Forested systems can operate on a year-round basis, except in extremely cold weather and require minimal storage. Some minimal storage (≥ 5 d) is recommended for operational flexibility in all situations.

The storage pond should be designed as an "in-line" component in the process because of the additional treatment benefits (particularly for nitrogen) which can occur. The algae which may develop in these ponds will be effectively removed by the soil component in the SR process. In contrast, storage ponds for the overland flow (OF) process should be an "off line" component due to the limited capability of algae removal in OF systems. The selection of a crop and the local climate conditions control the length of the operating season. Forested systems have the least constraints; perennial grasses permit a longer season than do agricultural row crops; application to parks and golf courses occur throughout the operating season for the facility, but at times of day when the public is not present.

The SR process can provide the best wastewater renovation potential as compared to all of the other "natural" systems (land treatment, aquaculture, wetlands, lagoons, etc). This is due to the physical, chemical and biological activity occurring as the wastewater percolates, at a slow rate, through the relatively fine textured soil in the SR process. The system can be utilized at marginal sites with poor soils, high groundwater tables, and steep slopes. The process not only provides an excellent quality effluent but can produce a marketable crop, and beneficially reuse the nutrients in the wastewater. Use of the wastewater for irrigation conserves other water resources for higher uses.

The suitability of a site for SR is limited by topography, soil type and depth, hydrogeologic conditions, climate, and availability of the land. The hydraulic loading rate on the system is typically limited by the permeability of the soil system or nitrate limitations on the receiving groundwater. In the latter case, the design is based on crop uptake and the nitrogen removal capacity of the soil-vegetation complex. The growing season for the crop selected, the time required for harvesting and maintenance, and periods of subfreezing (≤ 25°F) all limit the available operational period during each year. The availability of suitable land at a acceptable cost and within reasonable proximity of the wastewater sources can be a major limitation on adoption of the concept, and this issue must be considered during the

reasonable proximity of the wastewater sources can be a major limitation on adoption of the concept, and this issue must be considered during the preliminary planning stage. Most states have regulations or guidelines regarding preapplication treatment, loading rates, percolate or groundwater quality, buffer zones, and public exposure which may limit applicability of the concept.

Effluent quality is excellent after a few feet of travel in the unsaturated soil profile. BOD and TSS removals from 90 to 99 percent can be routinely expected. Total nitrogen removal from 50 to 95 percent can be realized depending on the crop selected and the loading rate adopted. Phosphorus removal depends on soil characteristics in the profile and ranges from 80 to 99 percent for most fine textured soils. The removal of pathogenic bacteria and virus can be almost complete after a few feet of travel. Metals removal depends on adsorptive and precipitation reactions but can reach 90 percent or more even in a sandy soil. The removal of phosphorus, pathogens, metals and similar substances will continue as the percolate moves further through the soil profile.

The field area for treatment typically utilized for Type 1 systems ranges from 6 to 21 ha for each 1000 m^3/d of design flow, depending on the crop selected and the characteristics of the site. In colder climates with storage times extending for several months an additional 3 to 11 ha per 1000 m^3/d may be needed for the storage pond and for roads and buffer zones.

_ A very preliminary cost estimate for planning purposes can be derived using the following procedures.

1.Costs are based on November 1990 (ENR Index 4780), ($US).

2. Costs for include 75 day storage lagoon, Annual wastewater application of 3 m per year, clearing, grubbing, grading, and establishment of vegetation on the site, distribution system using solid set sprinklers at 13 units/ha, with buried distribution piping, or center pivots with maximum coverage at one unit per 50 ha.

3. O & M costs include replacement of sprinklers, pumps and valve controls, maintenance of storage lagoon, and underdrains when used.

6. Costs do not include preliminary treatment process, transmission to site, final disinfection, or any service roads or fencing.

8. Equations below are valid for preliminary estimates up to 4000 m^3/d. wastewater flow.

C = Costs in millions of dollars ($ U.S.)

Q = Wastewater flow in Mgal/d (US)

CONSTRUCTION COSTS	OPERATION AND MAINTENANCE COSTS
Sprinklers	Sprinklers
Solid Set	Solid Set

No Underdrains

$$C = 1.400(Q)^{0.682}$$

No Underdrains

$$C = 0.050(Q)^{0.592}$$

Sprinklers
Center Pivot
No Underdrains

$$C = 1.148(Q)^{0.682}$$

Sprinklers
Center Pivot
No Underdrains

$$C = 0.064(Q)^{0.592}$$

Potential Application in Ontario

It would be possible to construct and operate a SR land treatment system for lagoon polishing almost anywhere in Ontario, although the operating season might be very short in the north. It would be a seasonal operation anywhere in Ontario with storage of lagoon effluent required during the coldest part of the year.

If a sufficient area is available with suitable soils the SR process will provide better treatment than any of the other technologies discussed in this report. It does require a relatively large land area and more operation and maintenance attention than the wetland concepts. Although the SR concept is technically feasible for lagoon upgrading in Ontario it is not likely to be economically competitive with the other alternatives.

CONCLUSIONS

1. The use of intermittent sand filters, two types of constructed wetlands, rock filters, aquaculture, and slow rate land treatment have been considered in this report for use in Ontario for lagoon polishing.

2. Rock filters are considered to be unacceptable due to their marginal performance for BOD and SS and inability to deal effectively with NH_3 and H_2S.

3. The aquaculture and land treatment concepts are both technically feasible for application in Ontario but are unlikely to be economically competitive with the other alternatives.

4. The intermittent sand filter concept is technically feasible for use in Ontario. Capital costs and O & M requirements are higher for this approach as compared to constructed wetlands. This is at least partially offset by the much longer experience record, and therefore a higher confidence level in this technology. Ammonia removal during the coldest winter months may be marginal so seasonal operation of an intermittent sand filter may be necessary to achieve high water quality standards.

5. Both wetland concepts, as presently configured are less costly to build and operate than an intermittent sand filter. Seasonal operations in most of Ontario might be required to sustain high water quality standards in the final

effluent. Both types of wetlands, as presently configured, provide insufficient oxygen to support the desired levels of nitrification. A number of corrective measures are possible but these are likely to increase both the capital and O & M costs, with the total costs still somewhat less than intermittent sand filters.

6. The cost analyses presented in this report do not include the cost for land since that is highly variable and site specific. If land costs are high then the intermittent sand filter is probably the preferred technology since it requires significantly less land than the wetland concepts. If land costs are low, then the wetland may be preferred because the O & M requirements are much less than required for an intermittent sand filter.

7. Additional information on the treatment concepts discussed in this report can be found in the following references.

1. WPCF, (1990) MOP FD-16, Natural Systems for Wastewater Treatment, WPCF, Alexandria, VA.
2. US EPA, (1983). Design Manual - Municipal Wastewater Stabilization Ponds, EPA-625/1-83-015, US EPA CERI, Cincinnati, OH.
3. Reed, S.C., et al. (1988). Natural Systems for Waste Management and Treatment, McGraw Hill Book Co. New York, NY.
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5. Russell, J.S., E.J. Middlebrooks, J.H. Reynolds, (1980). Wastewater Stabilization Lagoon-Intermittent Sand Filter system, EPA 600/2-80-032, US EPA Cincinnati, OH.
6. US EPA, (1988). Design Manual - Constructed Wetlands and Aquatic Plant Systems for Municipal Wastewater Treatment, EPA 625/1-88/022, US EPA CERI, Cincinnati, OH.
7. IAWPRC, (1990) Constructed Wetlands in Water Pollution Control, Pergamon Press, New York, NY.
8. TVA/EPA, (1989) Constructed Wetlands for Wastewater Treatment - Municipal, Industrial and Agricultural, Lewis Publishers, Chelsea, MI.
9. Dornbush, J. N. (1991). Constructed Wastewater Wetlands The Answer in South Dakota's Challenging Environment, presented at: Constructed Wetlands for Water Quality Improvement - International Symposium, Pensacola, FL Oct 1991.
10. Middlebrooks, E.J. Review of Rock Filters for Upgrade of Lagoon Effluents, Jour. WPCF, 60(9), 1657-1662, September 1988.
11. U.S. EPA (1981). Process Design Manual Land Treatment of Municipal Wastewater, U.S. EPA CERI, Cincinnati, OH.
12. Reed, S.C. (1992) Solar Aquatics for Wastewater Treatment at Sugarbush Vermont, WPCF (in press, July 1992).

APPENDIX 4 - COSTING INFORMATION

The following appendix details cost breakdowns and costing assumptions which have been used throughout the report. Historical costs have been compiled from information provided by the various parties involved in the design and construction of the facility. In many cases, only total or partially broken down costs were provided for a particular plant, which did not necessarily fit into the major headings used herein (i.e. Mobilization/Demobilization, Site Work, etc.). For these instances, the authors have attempted to subdivide costs accordingly, but may not necessarily reflect the actual breakdown. Engineering costs, if not available, were assumed to be 15% of total construction. All historical costs noted have been adjusted to March 1992 dollars using an ENR Index of 6537.

TABLE A4.1

CAPITAL COST BREAKDOWN – EXISTING FACILITIES (1)

(MARCH 1992 ENR = 6537)

CAPITAL RECOVERY FACTOR = 0.10185
(FOR I=8%)

PLANT	DUTTON	RODNEY	COOKSTOWN	COLBORNE	STAYNER	SUTTON	TOTTENHAM	LINDSAY	SCHOMBERG	NEW HAMBURG
FLOW (M^3/D)	558	590	825	1375	1875	2046	2257	15870	683	2700
1 SITE WORK	$193,679	$193,679	$265,511				$553,667	$125,932	$461,074	
2 CONC. TANKS	$284,840	$284,840	$312,791				$255,060	$354,898		
3 EQUIPMENT	$337,759	$337,759	$222,338				$456,620	$789,935	$92,010	
4 PROC. PIPING	$246,021	$246,021	$93,812					$107,614	$59,427	
5 METALS	$125,869	$125,869	$12,442				$12,442	$45,793	$17,418	
6 BUILDINGS	$419,385	$419,385	$97,918				$88,338	$68,690	$153,691	
7 LAGOONS			$560,385		$2,363,970				$829,933	$576,794
8 ELECTRICAL	$168,736	$168,736	$141,838				$128,152	$61,621	$79,919	
9 C & I	$102,141	$102,141						$91,587		
10 MISC.	$184,920	$184,920	$137,110	$1,755,072	$3,578,650			$123,642	$88,116	$1,730,382
11 ENGINEERING (% engineering)	$592,416 29%	$592,416 29%	$253,980 14%	$255,618 15%	$1,022,471 17%		$223,955 15%	$217,518 12%	$245,906 14%	$346,076 15%
TOTAL CAPITAL	$2,655,766	$2,655,766	$2,098,126	$2,010,690	$6,965,091	n/a	$1,718,233	$1,987,431	$2,027,495	$2,653,253
NORM. CAPITAL (2)	$4,759	$4,501	$2,543	$1,462	$3,715		$761	$125	$2,969	$983
AMOR. CAPITAL (3)	$270,490	$270,490	$213,694	$204,789	$709,395		$175,002	$202,420	$206,500	$270,234
O/M COST	n/a	n/a	$87,450	$84,650	n/a	$112,520	$77,235	$283,000	$47,630	$120,000
NORM. & AMOR (4) (no o & m)	$485	$458	$259	$149	$378		$78	$13	$302	$100
NORM. & AMOR (4) (with o & m)	n/a	n/a	$341	$211	n/a	n/a	$112	$31	$372	$145

notes:

(1) – all costs in 1992 dollars
(2) – in \$/cubic metre of design flow.
(3) – in \$/year, amortized over 20 year period with an assumed interest rate of 8%.
(4) – in \$/year/cubic metre per day of design flow.

TABLE A4.2

OPERATIONS AND MAINTENANCE COSTS – EXISTING FACILITIES (1)

PLANT FLOW(M ^ 3/D)	DUTTON 558	RODNEY 590	COOKSTOW 825	COLBORNE 1375	STAYNER 1875	SUTTON 2046	TOTTENHAM 2257	LINDSAY 15870	SCHOMBER(683	NEW HAMBURG 2700
LABOUR			$21,330	$31,520		$40,820	$27,810	$57,050	$33,370	$21,000
MATERIALS			$9,310	$3,200		$17,700	$10,020	$7,000	$990	
CHEMICALS			$8,990	$11,000		$17,070	$15,140	$63,000	$0	
HYDRO			$18,170	$8,000		$18,930	$23,125	$100,000	$3,230	
MISCELLANEOU:			$9,650	$30,930		$18,000	$1,140	$55,950	$10,040	$99,000
TOTAL O & M	N/A	N/A	$67,450	$84,650	N/A	$112,520	$77,235	$283,000	$47,630	$120,000
TOTAL/M ^ 3 DESIGN			$82	$62		$55	$34	$18	$70	$44

notes: (1) – annual costs in 1992 dollars, ENR index = 6537

SUMMARY OF DESIGN ASSUMPTIONS USED FOR COSTING CONVENTIONAL LAGOONS

Continuous Discharge Facultative Lagoons

- lagoon sized for 60 days storage
- 1.8 m operating depth with 0.6 m freeboard
- 2 cells
- continuous feed chemical precipitant for phosphorus removal (costing based upon using Alum)
- small control building housing chemical feed equipment
- no artificial or synthetic liner in lagoon or berms (soil conditions condusive for lagoon construction)

Feed-Draw Facultative Lagoons

- lagoon sized for 180 days storage for seasonal discharge and 360 days storage for annual discharge
- 2 cells
- 1.8 m operating depth, with 0.6 m freeboard
- continuous feed chemical precipitant for phosphorus removal (costing based upon using Alum)
- small control building housing chemical feed equipment
- no artificial or synthetic liner in lagoon or berms (soil conditions conclusive for lagoon construction)

Aerated Lagoons

- lagoon sized for 30 day retention, including final settling zone
- continuous discharge
- aeration via diffused mat aeration system, using mechanical blowers
- continuous feed chemical precipitant (alum used for phosphorus removal
- no artificial or synthetic liner in lagoon or berms (soil conditions conclusive for lagoon construction)
- 1.8 m operating depth with 0.6 m freeboard
- control building to house blowers, chemical feed equipment

Aerated-Facultative Lagoons

- lagoons sized for 25 days total retention, including 5 days in aeration cell
- two cells (one aerated, one facultative)
- continuous discharge
- aeration via diffused mat aeration system, using mechanical blowers
- continuous feed chemical precipitant (alum) used for phosphorus removal
- no artificial or synthetic liner in lagoon or berms (soil conditions conclusive for lagoon construction)
- 1.8 m operating depth with 0.6 m freeboard
- control building to house blowers and chemical feed equipment

Capital and Operations/Maintenance costs breakdowns of the above conventional lagoons have been indicated in Tables A4.3 to A4.7.

Operations and maintenance costs were estimated from reviewing historical operations costs for 1989-1991 for a sample of approximately 50 existing conventional lagoons in Ontario. These were divided into fill-draw, aerated and aerated-facultative lagoons (see Figures A4.1 - A4.3). O & M costs for continuous discharge lagoons, which were not present in the sample, were estimated separately.

TABLE A4.3

LAGOON COST ESTIMATE
CONTINUOUS DISCHARGE LAGOONS

SIZE	300 (M ^3/D)	1000 (M ^3/D)	3300 (M ^3/D)
RETENTION TIME (D)	60	60	60
LAGOON DEPTH (M)	2.4	2.4	2.4
LAGOON VOLUME (M ^3)	18000	60000	198000
SURFACE AREA (M ^2)	7500	25000	82500
MOBILIZATION/ DEMOBILIZATION	$20,000	$30,000	$40,000
SITE PREP	$15,000	$20,000	$25,000
EXCAVATION/ BERM CONSTRUCTION	$112,500	$300,000	$825,000
CONTROL BLDG	$50,000	$50,000	$50,000
PIPING	$5,000	$10,000	$25,000
CHEMICAL SYSTEM	$40,000	$50,000	$50,000
SITE WORK	$30,000	$40,000	$50,000
SUB TOTAL	$272,500	$500,000	$1,065,000
CONTINGENCIES	$40,875	$75,000	$159,750
ENGINEERING	$47,006	$86,250	$183,713
TOTAL CAPITAL	$360,381	$661,250	$1,408,463
O&M COSTS	$25,000	$45,000	$80,000
AMORTIZED CAP + O&M COST	$61,705	$112,348	$223,452
PERCENT O&M	41%	40%	36%

TABLE A4.4

LAGOON COST ESTIMATE
FILL–DRAW LAGOONS (SEASONAL)

SIZE	300 (M^3/D)	1000 (M^3/D)	3300 (M^3/D)
RETENTION TIME (D)	180	180	180
LAGOON DEPTH (M)	2.4	2.4	2.4
LAGOON VOLUME (M^3)	54000	180000	594000
SURFACE AREA (M^2)	22500	75000	247500
MOBILIZATION/ DEMOBILIZATION	$30,000	$40,000	$50,000
SITE PREP	$20,000	$25,000	$30,000
EXCAVATION/ BERM CONSTRUCTION	$270,000	$750,000	$2,227,500
CONTROL BLDG	$50,000	$50,000	$50,000
PIPING	$10,000	$20,000	$50,000
CHEMICAL SYSTEM	$40,000	$50,000	$50,000
SITE WORK	$40,000	$60,000	$75,000
SUB TOTAL	$460,000	$995,000	$2,532,500
CONTINGENCIES	$69,000	$149,250	$379,875
ENGINEERING	$79,350	$171,638	$436,856
TOTAL CAPITAL	$608,350	$1,315,888	$3,349,231
O&M COSTS	$21,858	$39,908	$72,496
AMORTIZED CAP + O&M COST	$83,818	$173,931	$413,615
PERCENT O&M	26%	23%	18%

TABLE A4.5

LAGOON COST ESTIMATE
FILL–DRAW DISCHARGE LAGOONS (ANNUAL)

SIZE	300 (M^3/D)	1000 (M^3/D)	3300 (M^3/D)
RETENTION TIME (D)	360	360	360
LAGOON DEPTH (M)	2.4	2.4	2.4
LAGOON VOLUME (M^3)	108000	360000	1188000
SURFACE AREA (M^2)	45000	150000	495000
MOBILIZATION/ DEMOBILIZATION	$30,000	$40,000	$50,000
SITE PREP	$20,000	$25,000	$30,000
EXCAVATION/ BERM CONSTRUCTION	$405,000	$1,200,000	$3,465,000
CONTROL BLDG	$50,000	$50,000	$50,000
PIPING	$20,000	$25,000	$30,000
CHEMICAL SYSTEM	$40,000	$50,000	$50,000
SITE WORK	$40,000	$60,000	$75,000
SUB TOTAL	$605,000	$1,450,000	$3,750,000
CONTINGENCIES	$90,750	$217,500	$562,500
ENGINEERING	$104,363	$250,125	$646,875
TOTAL CAPITAL	$800,113	$1,917,625	$4,959,375
O&M COSTS	$21,858	$39,908	$72,496
AMORTIZED CAP + O&M COST	$103,349	$235,218	$577,608
PERCENT O&M	21%	17%	13%

TABLE A4.6

LAGOON COST ESTIMATE
AERATED LAGOONS

SIZE	300 (M^3/D)	1000 (M^3/D)	3300 (M^3/D)
RETENTION TIME (D)	30	30	30
LAGOON DEPTH (M)	2.4	2.4	2.4
LAGOON VOLUME (M^3)	9000	30000	99000
SURFACE AREA (M^2)	3750	12500	41250
MOBILIZATION/ DEMOBILIZATION	$15,000	$20,000	$30,000
SITE PREP	$15,000	$20,000	$25,000
EXCAVATION/ BERM CONSTRUCTION	$63,750	$200,000	$618,750
AERATION SYS	$180,000	$400,000	$975,000
CONTROL BLDG	$50,000	$75,000	$75,000
PIPING	$20,000	$25,000	$30,000
CHEMICAL SYSTEM	$40,000	$50,000	$50,000
SITE WORK	$25,000	$35,000	$45,000
SUB TOTAL	$408,750	$825,000	$1,848,750
CONTINGENCIES	$61,313	$123,750	$277,313
ENGINEERING	$70,509	$142,313	$318,909
TOTAL CAPITAL	$540,572	$1,091,063	$2,444,972
O&M COSTS	$31,177	$56,920	$103,402
AMORTIZED CAP + O&M COST	$86,234	$168,045	$352,422
PERCENT O&M	36%	34%	29%

TABLE A4.7

LAGOON COST ESTIMATE
AERATED–FACULTATIVE LAGOONS

SIZE	300 (M^3/D)	1000 (M^3/D)	3300 (M^3/D)
RETENTION TIME (D)	25	25	25
LAGOON DEPTH (M)	2.4	2.4	2.4
LAGOON VOLUME (M^3)	7500	25000	82500
SURFACE AREA (M^2)	3125	10417	34375
MOBILIZATION/ DEMOBILIZATION	$15,000	$20,000	$30,000
SITE PREP	$15,000	$20,000	$25,000
EXCAVATION/ BERM CONSTRUCTION	$53,125	$166,667	$515,625
AERATION SYS	$90,000	$200,000	$500,000
CONTROL BLDG	$50,000	$75,000	$75,000
PIPING	$20,000	$25,000	$30,000
CHEMICAL SYSTEM	$40,000	$50,000	$50,000
SITE WORK	$25,000	$35,000	$45,000
SUB TOTAL	$308,125	$591,667	$1,270,625
CONTINGENCIES	$46,219	$88,750	$190,594
ENGINEERING	$53,152	$102,063	$219,183
TOTAL CAPITAL	$407,495	$782,479	$1,680,402
O&M COSTS	$10,374	$36,726	$128,650
AMORTIZED CAP + O&M COST	$51,877	$116,422	$299,799
PERCENT O&M	20%	32%	43%

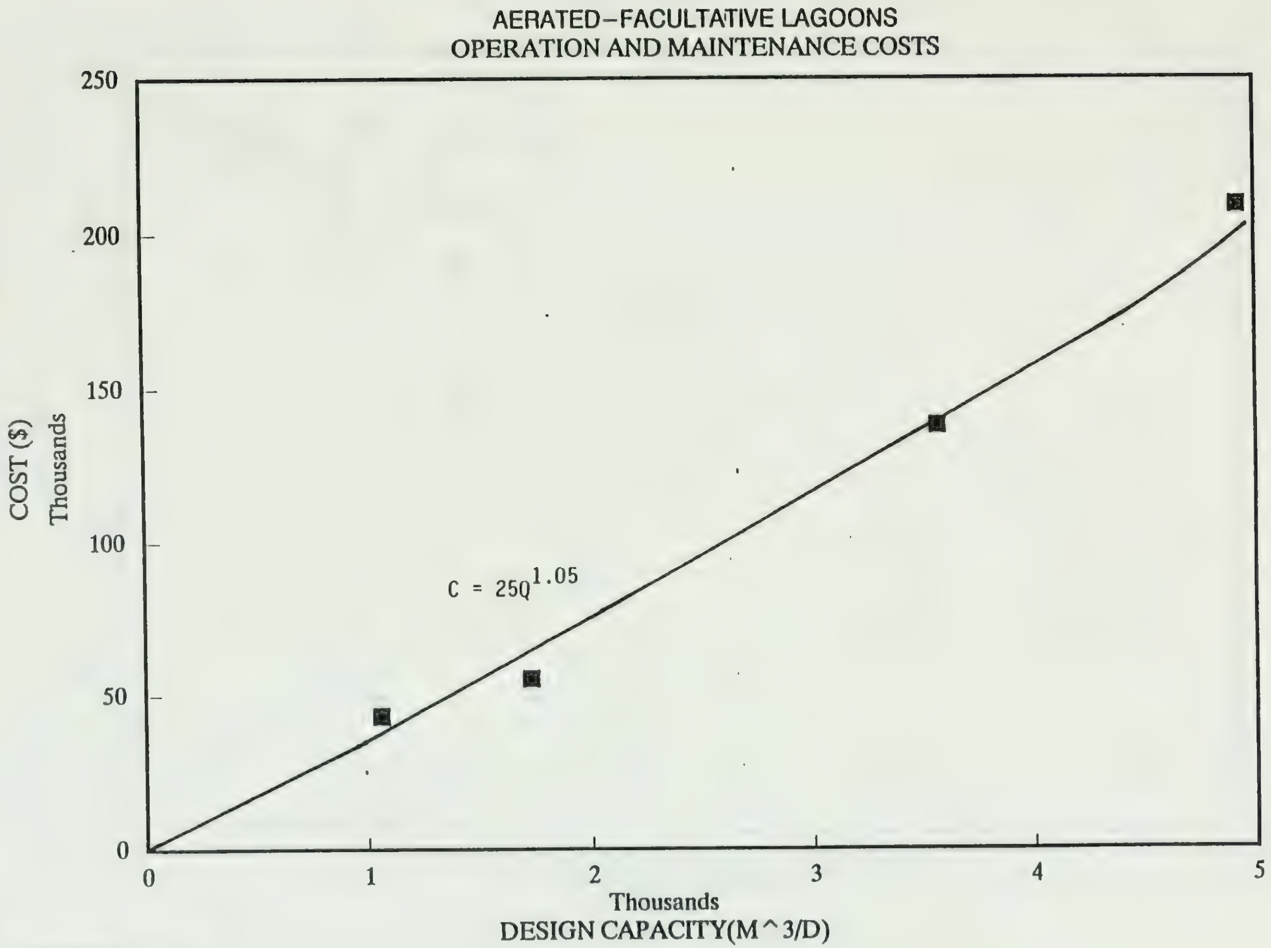

AERATED-FACULTATIVE LAGOONS
OPERATION AND MAINTENANCE COSTS
250
200
150
100
50
0
COST ($)
Thousands
C = 25Q^1.05
0
1
2
3
4
5
Thousands
DESIGN CAPACITY(M^3/D)

FIGURE A4.3

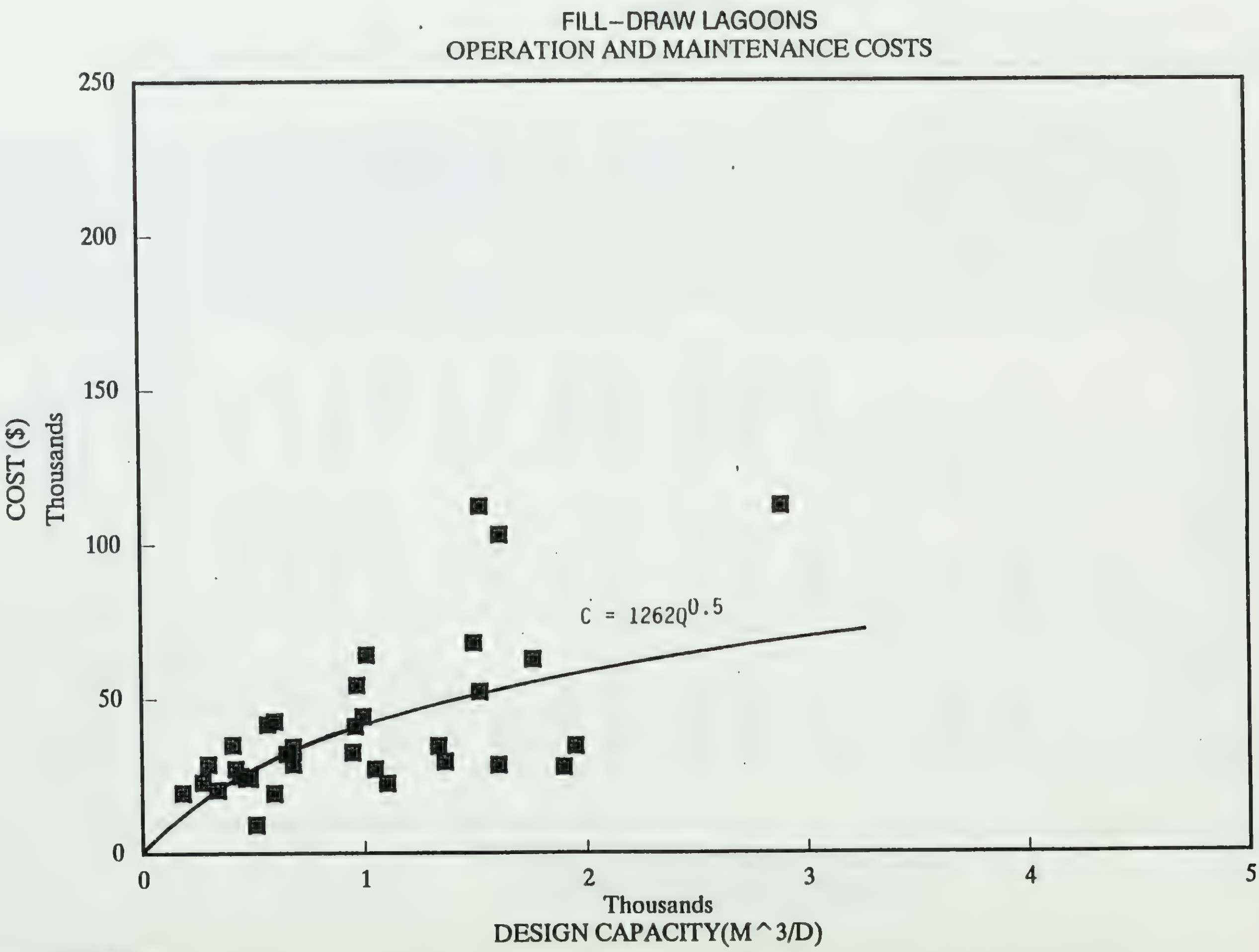
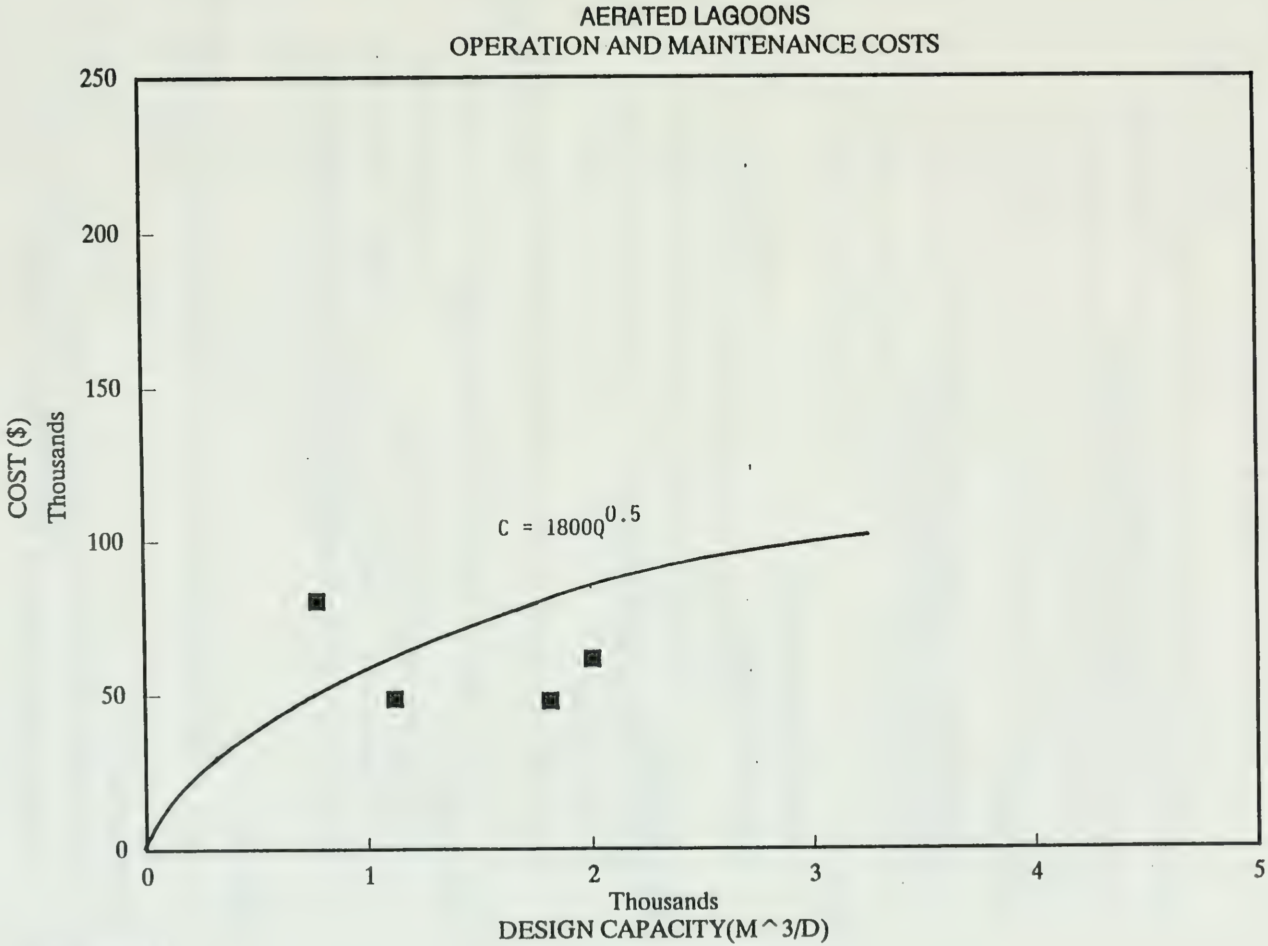

FILL--DRAW LAGOONS
OPERATION AND MAINTENANCE COSTS
COST ($)
Thousands
250
200
150
100
50
0
C = 1262Q0.5
0
1
2
3
4
5
Thousands
DESIGN CAPACITY(M^3/D)

FIGURE A4.1



AERATED LAGOONS
OPERATION AND MAINTENANCE COSTS
COST ($)
Thousands
250
200
150
100
50
0
C = 1800Q^0.5
0
1
2
3
4
5
Thousands
DESIGN CAPACITY(M^3/D)

FIGURE A4.2

SUMMARY OF DESIGN ASSUMPTIONS USED FOR COSTING
SUTTON PLANTS

Preliminary Treatment

- 2 manually raked bar screens in concrete channel
- 2 Grit channels with sutural weirs

Aeration Tanks

- 2 earthen basins with Rip/Rap sides, or 2 concrete tanks
- bridge mounted mechanical surface aeration

Clarifiers

- 2 circular concrete clarifiers
- scum collection system

Control Building

- sized for office, lab and washroom
- full time operator
- housing for return sludge pumping in basement, and chemical feed pumping

Chemical System

- 2 chemical feed pumps (1 duty, 1 standby)
- cost of system based on using Alum
- outdoor storage tank, heat traced, with minimum 5000 gallons capacity

Equipment

- return sludge pumping system, with 2 return sludge pumps (1 duty, 1 standby)

Flow Measurement

- flow measurement using flumes with ultra-sonic device
- measure both raw sewage and effluent discharge to lagoons

Lagoon Costs

A. Retrofit Facilities

- existing lagoon has capacity for continuous discharge (60 days storage)
- includes cost of berming off 1 existing lagoon to create 2 cells
- for sludge discharge to lagoons, includes cost to excavate sludge storage "pod" in each lagoon, sized to hold 5 years worth of sludge at 10% solids concentration, sludge haulage cost includes dredging and hauling contents of pod every 5 years
- for sludge discharge to onsite sludge storage lagoon, cost includes berming off portion of existing lagoon system to create sludge storage lagoon sized for storage at 4% solids concentration

B. New (Green Field) Facilities

- lagoon sized for continuous discharge (60 days storage)
- sludge storage pod (for sludge discharge to lagoons) as in A
- separate lagoon constructed for the sludge discharge to separate onsite lagoon scenario

Sludge Holding Tank

- required for sludge discharge to separate onsite lagoon scenario
- concrete construction
- sized for 45 days sludge age
- supernating capability to achieve 4% solids
- aspirating mixer to provide mixing and aeration

Disinfection

- not required

Process Piping

- cast/ductile iron piping for return sludge pumps

Electrical & Controls

- 15% of construction cost (excluding any lagoon costs)
- local controls only

Engineering

- 15% of total construction

SUMMARY OF DESIGN ASSUMPTIONS USED FOR COSTING NEW HAMBURG PLANTS

Preliminary Treatment

- none provided

Control Building

- small office, lab facilities and washroom
- part time operator
- housing for effluent pumps, electrical and control panels

Chemical System

- 2 chemical feed pumps (1 duty, 1 standby)
- cost of system based upon using Alum
- outdoor storage tank, heat traced, with minimum 5000 gallons capacity

Flow Measurement

- measure raw sewage flow with flume and filter effluent with Venturi

Lagoons

A. Retrofit Facilities
   - existing lagoon has capacity for seasonal discharge (180 days storage)

B. New (Green Field) Facilities
   - lagoon sized for seasonal discharge (180 days storage)

Filters

- liquid application to filter bed via distribution header
- graded filter media comprised of the following (as per New Hamburg):

- 760 mm of sand on top, followed by 75 mm of 5 mm crushed stone, 75 mm of 9.5 crushed stone, 75 mm of 13.2 mm crushed stone, and 75 mm of 19.0 mm crushed stone
- underdrain system via 100 mm perforated PVC pipe
- 2 or more filters to provide for standby capacity

Disinfection

- not required

Electrical & Controls

- 15% of construction cost (excluding lagoon cost)
- local controls only

Engineering

- 15% of total construction

TABLE A4.8

SUTTON PROCESS PLANTS

COST ESTIMATES

ITEM	RETROFIT FACILITIES WITH SLUDGE DISCHARGE TO LAGOONS			RETROFIT FACILITIES WITHOUT SLUDGE DISCHARGE TO LAGOONS			NEW FACILITIES WITH SLUDGE DISCHARGE TO LAGOONS			NEW FACILITIES WITHOUT SLUDGE DISCHARGE TO LAGOONS		
	300 (M^3/D)	1000 (M^3/D)	3300 (M^3/D)	300 (M^3/D)	1000 (M^3/D)	3300 (M^3/D)	300 (M^3/D)	1000 (M^3/D)	3300 (M^3/D)	300 (M^3/D)	1000 (M^3/D)	3300 (M^3/D)
1 MOB/DEMOB	$30,000	$40,000	$50,000	$30,000	$40,000	$50,000	$40,000	$65,000	$80,000	$40,000	$65,000	$80,000
2 SITE WORK	$150,000	$175,000	$200,000	$150,000	$175,000	$200,000	$225,000	$275,000	$325,000	$225,000	$275,000	$325,000
3 INLET WORKS	$25,000	$40,000	$60,000	$25,000	$40,000	$60,000	$25,000	$40,000	$60,000	$25,000	$40,000	$60,000
4 AERATION TANKS	$80,500	$160,600	$331,900	$80,500	$160,600	$331,900	$80,500	$160,600	$331,900	$80,500	$160,600	$331,900
5 CLARIFIERS	$135,900	$257,000	$499,400	$135,900	$257,000	$499,400	$135,900	$257,000	$499,400	$135,900	$257,000	$499,400
6 CONTROL BUILDING	$119,300	$119,300	$119,300	$119,300	$119,300	$119,300	$119,300	$119,300	$119,300	$119,300	$119,300	$119,300
7 CHEMICAL SYSTEM	$56,000	$63,000	$73,000	$56,000	$63,000	$73,000	$56,000	$63,000	$73,000	$56,000	$63,000	$73,000
8 EQUIPMENT	$30,000	$36,000	$48,000	$30,000	$36,000	$48,000	$30,000	$36,000	$48,000	$30,000	$36,000	$48,000
9 LAGOON COSTS	$7,896	$19,036	$56,150	$3,650	$5,800	$11,200	$140,396	$349,036	$931,150	$133,000	$331,000	$877,500
10 SLUDGE HOLDING TANK				$47,000	$68,000	$107,000				$47,000	$68,000	$107,000
11 PROCESS PIPING	$140,000	$200,000	$275,000	$140,000	$200,000	$275,000	$140,000	$200,000	$275,000	$140,000	$200,000	$275,000
12 ELECTRICAL & CONTROLS	$76,670	$109,090	$165,660	$81,370	$115,890	$176,360	$85,170	$121,590	$181,160	$89,870	$128,390	$191,860
13 CONTINGENCIES	$127,690	$182,854	$281,762	$134,808	$192,089	$292,674	$161,590	$252,979	$438,587	$168,236	$261,494	$448,194
14 ENGINEERING	$146,843	$210,282	$324,026	$155,029	$220,902	$336,575	$185,828	$290,926	$504,374	$193,471	$300,718	$515,423
TOTAL CAPITAL	$1,125,799	$1,612,162	$2,484,197	$1,188,557	$1,693,580	$2,580,409	$1,424,684	$2,230,431	$3,866,871	$1,483,276	$2,305,501	$3,951,577
YEARLY O&M COST (w/o sludge disp)	$50,000	$80,000	$150,000	$50,000	$80,000	$150,000	$50,000	$80,000	$150,000	$50,000	$80,000	$150,000
YEARLY SLUDGE DISPOSAL COST	$5,000	$10,000	$20,000	$2,190	$5,840	$21,900	$5,000	$10,000	$20,000	$2,190	$5,840	$21,900
AMORTIZED CAP + O&M COST	$169,663	$254,199	$423,015	$173,245	$258,331	$434,715	$200,104	$317,169	$563,841	$203,262	$320,655	$574,368

TABLE A4.9

NEW HAMBURG PROCESS PLANTS

COST ESTIMATES

ITEM	RETROFIT FACILITIES 300 (M^3/D)	RETROFIT FACILITIES 1000 (M^3/D)	RETROFIT FACILITIES 3300 (M^3/D)	NEW FACILITIES 300 (M^3/D)	NEW FACILITIES 1000 (M^3/D)	NEW FACILITIES 3300 (M^3/D)
1 MOB/DEMOB	$30,000	$40,000	$50,000	$30,000	$40,000	$50,000
2 SITE WORK	$135,000	$150,000	$170,000	$200,000	$240,000	$305,000
3 FILTERS	$28,400	$86,300	$285,000	$28,400	$86,300	$285,000
4 CONTROL BUILDING	$70,000	$70,000	$70,000	$70,000	$70,000	$70,000
5 CHEMICAL SYSTEM	$46,000	$48,000	$53,000	$46,000	$48,000	$53,000
6 EQUIPMENT	$22,000	$25,000	$30,000	$22,000	$25,000	$30,000
7 LAGOON COSTS				$300,000	$795,000	$2,307,500
8 PROCESS PIPING	$32,000	$40,000	$58,000	$32,000	$40,000	$58,000
9 ELECTRICAL & CONTROLS	$36,340	$45,930	$71,600	$42,840	$54,930	$85,100
10 CONTINGENCIES	$59,961	$75,785	$118,140	$115,686	$209,885	$486,540
11 ENGINEERING	$68,955	$87,152	$135,861	$133,039	$241,367	$559,521
TOTAL CAPITAL	$528,656	$668,167	$1,041,601	$1,019,965	$1,850,482	$4,289,661
YEARLY O&M COST	$35,000	$50,000	$90,000	$35,000	$50,000	$90,000
AMORTIZED CAP + O&M	$88,844	$118,053	$196,087	$138,883	$238,472	$526,902

Statistical Procedures for Setting Effluent Limits for Municipal STPs Using Monthly Average Data (from 1986 to 1988)

<u>For Monthly Limit</u>

Group the STPs using similar treatment technologies or produce similar effluent qualities.

Let X_{ij} = three years monthly averages for plant i and month j;
i= 1,.....,N where N is total number of STPs in the data base;
j= 1,...., 36 for 36 months of monthly averages

M_i = arithmetic mean of X_{ij}

= $(S_{j=1}^{36} X_{ij})/36$

where $S^{j=1\ 36}$ = sum for the 36 monthly averages

S_{Mi} = standard deviation of X_{ij}

= $\mathrm{SQRT}(S_{j=1}^{36} (x_{ij} - M_i)^2/35)$

where SQRT = square root

Then

LTA = Long Term average

= median of (M_i, i= 1, ...,N)

$V_{95,i}$= variability factor, 95th %-tile for plant i

= $(M_i + 1.64 \times S_{Mi})/M_i$

V_{95} = arithmetic mean of $V_{95,i}$ for i = 1, ...,N

= $(S_{i=1}^{N} V_{95,i})/N$

where $S_{i=1}^{N}$ = sum for the N number of plants

Monthly Limit (ML) is calculated as

ML = LTA x V_{95}

For Annual Limit

X_{ik} = three years annual averages for plant i and year k where k = 1, 2, 3; i = 1, ..., N for N number of STPs

A_i = arithmetic mean of X_{ik} (=M_i)

= $(S_{k=1}{}^{3}\ X_{ij})/3$

where $S_{k=1}{}^{3}$ = sum for the 3 years annual averages

S_{Ai} = standard deviation of X_{ij}/SQRT(12)

= $SQRT((S_{j=1}{}^{36}\ (X_{ij} - M_i)^2/36)/SQRT(12))$

= S_{Mi} / SQRT(12)

Then

LTA = long term average

= median of (A_i, i = 1, ..., N)

$V_{95,i}$= variability factor 95th %-tile for plant i

= $(A_i + 1.64 \times S_{Ai})/A_j$

V_{95} = arithmetic mean of $V_{95,i}$ for i = 1,...,N

= $(S_{i=1}{}^{N}\ V_{95,i})/N$

Annual Limit (AL) is calculated as

AL = LTA x V_{95}

Daily Limit

LTA = long term average for all plants
= median of (M_i, i = 1, ..., N)

n_i = sample size per month plant i

1. If daily data are normally distributed, then

$V_{99,i}$ = variability factor 99th %-tile for plant i

= $(M_i + 2.326 \times S_{Mi} \times SQRT(n_i)/M_i$

V_{99} = variability factor 99th %-tile for all plants

= arithmetic mean of $V_{99,i}$ for i = 1, ..., N

= $(S_{i=1}{}^{N} V_{99,i})/N$

Daily Limit (DL) is calculated as

DL = LTA x V_{99}

2. If daily data are lognormally distributed, then

$V_{99,i}$= $\exp\{2.236 \times Sd - 0.5 \times Sd^2\}$

where

Sd^2 = $\ln\{1 + (\exp\{Sd_{Mi}{}^2\} - 1) \times n_i\}$

V_{99} = variability factor 99th %-tile for all plants
= arithmetic mean of $V_{99,i}$ for i = 1,...., N)

= $(S_{i=1}{}^{N} V_{99,i})/N$

Daily Limit (DL) is calculated as

DL = LTA x V_{99}

North Continuous No Phosphorus Removal

BOB'S LAKE LAGOON
EARLTON LAGOON

North Continuous Phosphorus Removal

ENGLEHART LAGOON

North Annual No Phosphorus Removal

NONE

North Annual Phosphorus Removal

GLACKMEYER LAGOON

North Seasonal No Phosphorus Removal

BARWICK (HAMLET OF)
BELLE VALLEE LAGOON
BRUCE MINES LAGOON
BURKS FALLS LAGOON
CECILE TRAILER PARK LAGOON
CHELMSFORD LAGOON
COCHENOUR LAGOON
EMO LAGOON
FAUQUIER LAGOON
GORE BAY LAGOON
LITTLE CURRENT LAGOON
MANITOWANING LAGOON
MATTICE LAGOON
MOONBEAM LAGOON
NOELVILLE LAGOON
PORQUIS JUNCTION LAGOON
RAINY RIVER LAGOON
THESSALON LAGOON
WAHNAPITAE LAGOON
WEBBWOOD LAGOON

North Seasonal Phosphorus Removal

BLACK RIVER RAMORE (OP MAY 90)
CALLANDER LAGOON
DESBARATS
HALLEBOURG LAGOON
HARTY LAGOON
JOGUES LAGOON
KILLARNEY LAGOON
OPASATIKA LAGOON
POWASSAN LAGOON
ST.CHARLES LAGOON
SUNDRIDGE LAGOON
TEMAGAMI LAGOON
VAL-RITA LAGOON
VERNER LAGOON
WAWA LAGOON
WARREN LAGOON

North Aerated Lagoon No Phosphorus Removal

CHAPLEAU LAGOON
MANITOUWADGE LAGOON

North Aerated Lagoon Phosphorus Removal

NONE

North Aerobic Facultative Lagoon No Phosphorus Removal

HEARST LAGOON
NORTH GOWARD LAGOON

North Aerobic Facultative Lagoon Phosphorus Removal

NEW LISKEARD LAGOON
NORTH COBALT LAGOON

South Continuous No Phosphorus Removal

AMHERSTVIEW LAGOON
CARDIFF LAGOON
ELMVALE LAGOON
SCHOMBERG LAGOON (OP.JAN 90)
STEVENSVILLE-DOUGLASTOWN
LAGOON
WINGHAM LAGOON

South Continuous Phosphorus Removal

PERTH LAGOON
WIARTON LAGOON

South Annual No Phosphorus Removal

ALFRED LAGOON
EDWARDBURG
HENSALL LAGOON
OSWEGO PARK LAGOON
RUSSELL LAGOON
ST.ISIDORE LAGOON
THEDFORD LAGOON
WILLIAMSBURG PV LAGOON

South Annual Phosphorus Removal

BRIGDEN P.V. LAGOON
DRAYTON LAGOON (MUN OP JAN 1 1989)
MAXVILLE WASTE STAB POND (OP SEP 89)
MERLIN PV LAGOON
OIL SPRINGS LAGOON

South Seasonal No Phosphorus Removal

BRIGHT'S GROVE LAGOON
CASSELMAN LAGOON
CHESTERVILLE LAGOON
EMBRUN P.V.LAGOON
FOREST LAGOON
GANANOQUE LAGOON
LANCASTER LAGOON
LUCAN LAGOON
PLANTAGENET LAGOON
SMITHVILLE LAGOON
VANKLEEK HILL LAGOON
WINCHESTER LAGOON
ZURICH LAGOON

South Seasonal Phosphorus Removal

ARTHUR LAGOON
BEAVERTON RIVER 2 LAGOON (CANNINGTON)
BELMONT LAGOON
BLENHEIM LAGOON
COMBER LAGOON
COTTAM LAGOON
EDGEWATER BEACH LAGOON
ESSEX LAGOON N.E.
ESSEX LAGOON S.W.
GLENCOE LAGOON
GRAND BEND LAGOON
HARROW LAGOON
HAVELOCK LAGOON
HOLLAND LANDING LAGOON
JARVIS LAGOON
KINGSVILLE LAGOON
LAKE SIMCOE LAGOON (BEAVERTON)
LAKEFIELD LAGOON
MADOC LAGOON
MITCHELL'S BAY LAGOON
NEUSTADT LAGOON
NONQUON RIVER LAGOON (PORT PERRY)
NORWICH LAGOON
OIL CITY LAGOON
PARKHILL LAGOON
PORT LAMBTON LAGOON
PORT ROWAN LAGOON
PORT STANLEY LAGOON
SEAFORTH LAGOON
SOMBRA LAGOON
STELCO IND. PARK LAGOON
STIRLING LAGOON
STONEY POINT P.V. LAGOON
TARA LAGOON
TILBURY LAGOON
TOWNSEND LAGOON
TWEED LAGOON
WARKWORTH LAGOON
WATFORD LAGOON
WEST LORNE LAGOON
WESTPORT LAGOON
RIDGETOWN LAGOON

South Aerated Lagoon No Phosphorus Removal

HARRISTON LAGOON

South Aerated Lagoon Phosphorus Removal

COLCHESTER SOUTH TWP. (OP APRIL 89)
STRATHROY LAGOON

South Aerobic Facultative Lagoon No Phosphorus Removal

BIGGAR LAGOON
NIAGARA-ON-THE-LAKE LAGOON
ROCKLAND LAGOON

South Aerobic Facultative Lagoon Phosphorus Removal

ALEXANDRIA LAGOON
ALMONTE LAGOON
BRIGHTON LAGOON
KINCARDINE LAGOON
MILVERTON LAGOON
PLATTSVILLE LAGOON
TAVISTOCK LAGOON
WATERFORD LAGOON